BRIDGE STUDY—CLAIRMONT ROAD
NORTH PEACHTREE CREEK

A THESIS

Submitted in partial fulfillment

of the requirements for the Degree

of Master of Science in Civil Engineering

by

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Georgia School of Technology

Atlanta, Georgia

1940

Approved by

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PREFACE AND ACKNOWLEDGEMENTS

The following designs were made in compliance with the requirements for a thesis in the Department of Civil Engineering of the Georgia School of Technology and under the supervision of Professors F. C. Snow and J. M. Smith.

My grateful acknowledgements are due to Professors F. C. Snow and J. M. Smith for their helpful suggestions and criticism during the preparation of this thesis; to Mr. A. M. McCoy of Calvert Iron Works Inc., and to the Beers Construction Company for information concerning the cost of the material and construction. The kindness of the Georgia State Highway Department in making available certain information is acknowledged in detail in the course of the thesis.

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ILLUSTRATIONS

PART I

GENERAL INFORMATION

Introduction

The problem considered in this thesis is the design and comparison from the stand point of first cost of two types of bridges suitable to carry traffic on Clairmont Road over North Peachtree Creek.

The total span of this bridge is to be one hundred and thirty feet and the clear width of roadway twenty four feet (an increase of four feet over the present width of road).

References were made throughout the design and preparation of this thesis to the following books: "Highway Bridges" by J. E. Kirkham, "Bridge Engineering", Volume I and II, by Waddell, "Structural Theory" by Sutherland and Bowman, "Design of Highway Bridges" by Ketchum, and "Concrete Design Notes" by Professor F. C. Snow.

Information as to the soundings to determine the character and type of the foundation was made available to me through the kindness of the Georgia State Highway Department. This information is given on the profile sheet fig. 1 and 2

Due to the location of the road these bridges have been designed for class "A" loading. This loading is for bridges carrying normally heavy traffic units and the occasional passage of especially heavy loads. In both designs the specifications of the Georgia State Highway Department were followed throughout.

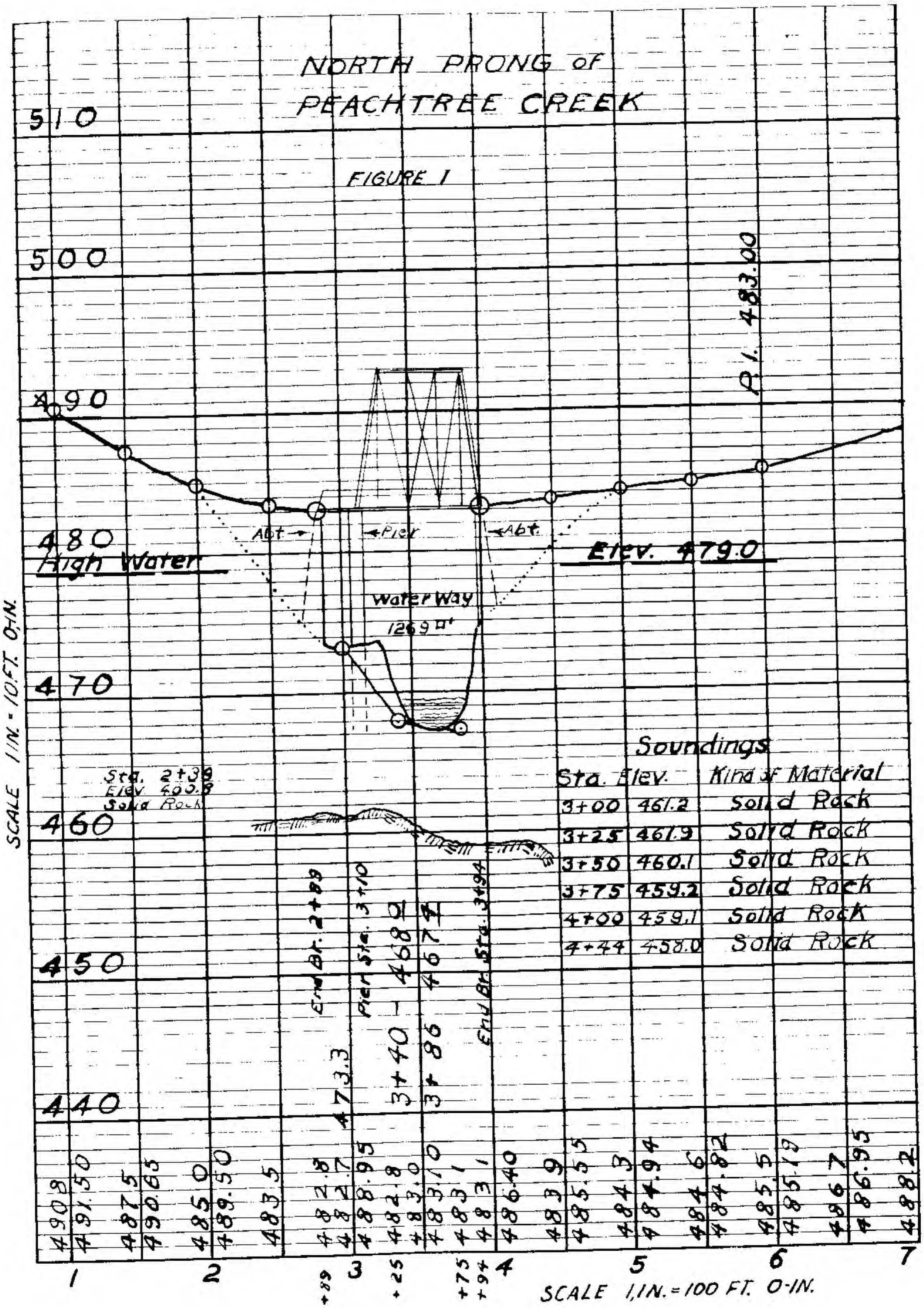
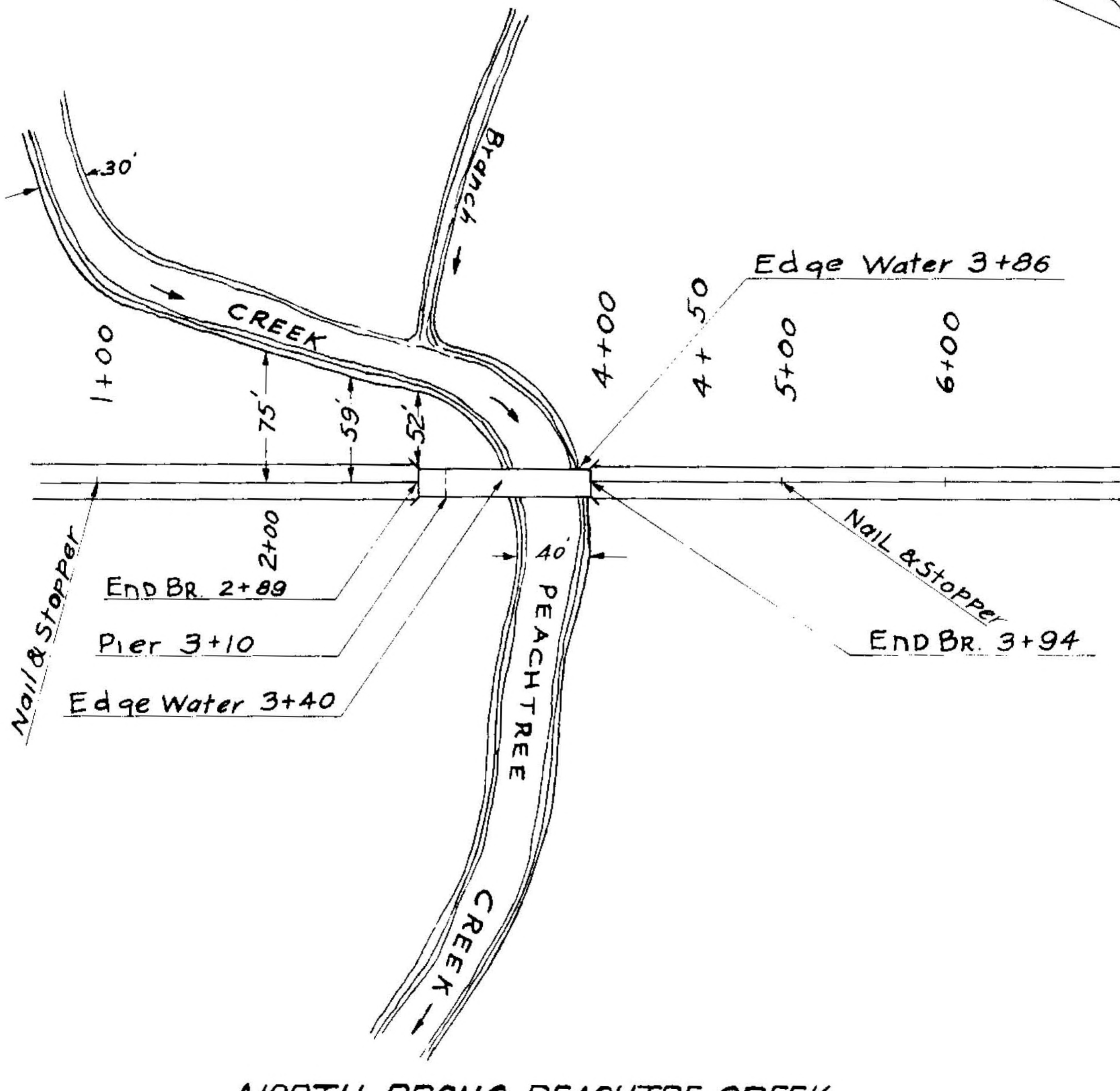

NORTH PRONG of
PEACHTREE CREEK
FIGURE I
510
500
490
480
470
460
450
440
P.I. 483.00
High Water
Elev. 479.0
Abt.
Pier
Abt.
Water Way
1269 □'
Sta. 2+38
Elev. 460.8
Solid Rock
Soundings
Sta. Elev. Kind of Material
3+00 461.2 Solid Rock
3+25 461.9 Solid Rock
3+50 460.1 Solid Rock
3+75 459.2 Solid Rock
4+00 459.1 Solid Rock
4+44 458.0 Solid Rock
End Br. 2+89
Pier Sta. 3+10
End Br. Sta. 3+94
473.3
3+40 - 468.0
3+86 - 467.4
490.8
491.50
487.5
490.65
485.0
489.50
483.5
482.8
482.7
488.95
482.8
483.0
483.10
483.1
483.1
486.40
483.9
485.55
484.3
484.94
484.6
484.82
485.5
485.19
486.7
486.95
488.2
1
2
+89
3
+25
+75
+94
4
5
6
7
SCALE 1 IN. = 10 FT. 0 IN.
SCALE 1 IN. = 100 FT. 0 IN.

FIGURE 2



NORTH PRONG PEACHTRE CREEK
SCALE 1.IN. = 100 FT

Standard Notation

E_s Modulus of Elasticity of Steel

E_c Modulus of Elasticity of Concrete

n $\frac{E_s}{E_c}$

f_s Tensile unit stress in reinforcement

f_c Compressive unit stress in concrete

M Bending moment

A_s Cross-sectional area of reinforcement

b width

d Effective depth, distance from compression side in concrete to center of reinforcement in tension

d' Distance from center of gravity of the tensile steel to tension side of beam

k Ratio of depth of neutral axis to Effective depth

j Ratio of lever arm of resisting couple to Effective depth

V Total shear

v Unit shearing stress

u Bond stress per unit area of bar surface

o Perimeter of bar

DL Dead Load

LL Live Load

IL Impact Load

L Span

E Effective width (or a width over which a load of one wheel is considered to be distributed)

W Total weight

w weight per foot

PI Point of Inflection

Description of Bridges

The following designs were made:

Part II. The Encased Steel "I" beam bridge composed of three thirty foot spans and one forty foot span.

Part III. A low Warren type through truss composed of two sixty five foot spans.

Description of loading

The structure is designed for the following loads:

(a) Dead loads which consists of the weight of the structure complete.

(b) Live load consists of a train of motor trucks hereinafter described for class "A" loading.

(c) Impact or dynamic effect of this live load.

(d) Lateral forces or the wind force on the structure.

Description of Class "A" Loading

Class "A" or H 15 loading consist of one truck of the gross weight indicated by the loading classification in tons followed by and preceded by trucks having a gross weight of three fourths of the gross weight indicated by the loading classification. The distances between the axles and wheels of trucks and the loads per axle are given in Fig. 3.

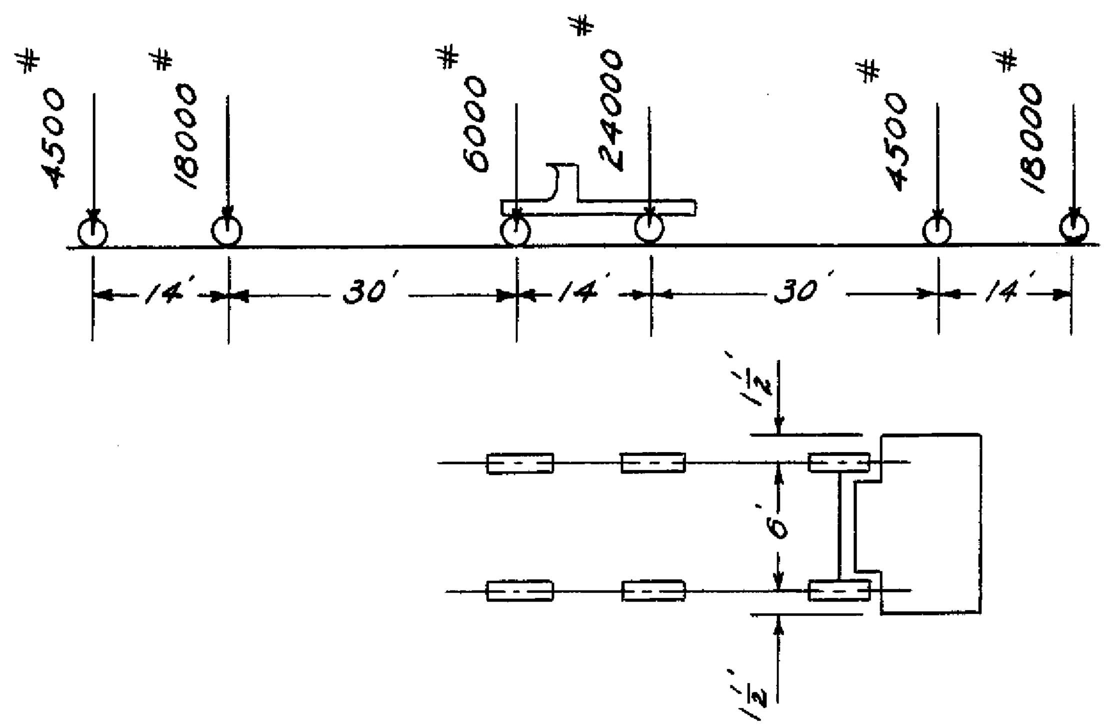


Figure 3

PART II

DESIGN OF ENCASED STEEL I BEAM BRIDGE

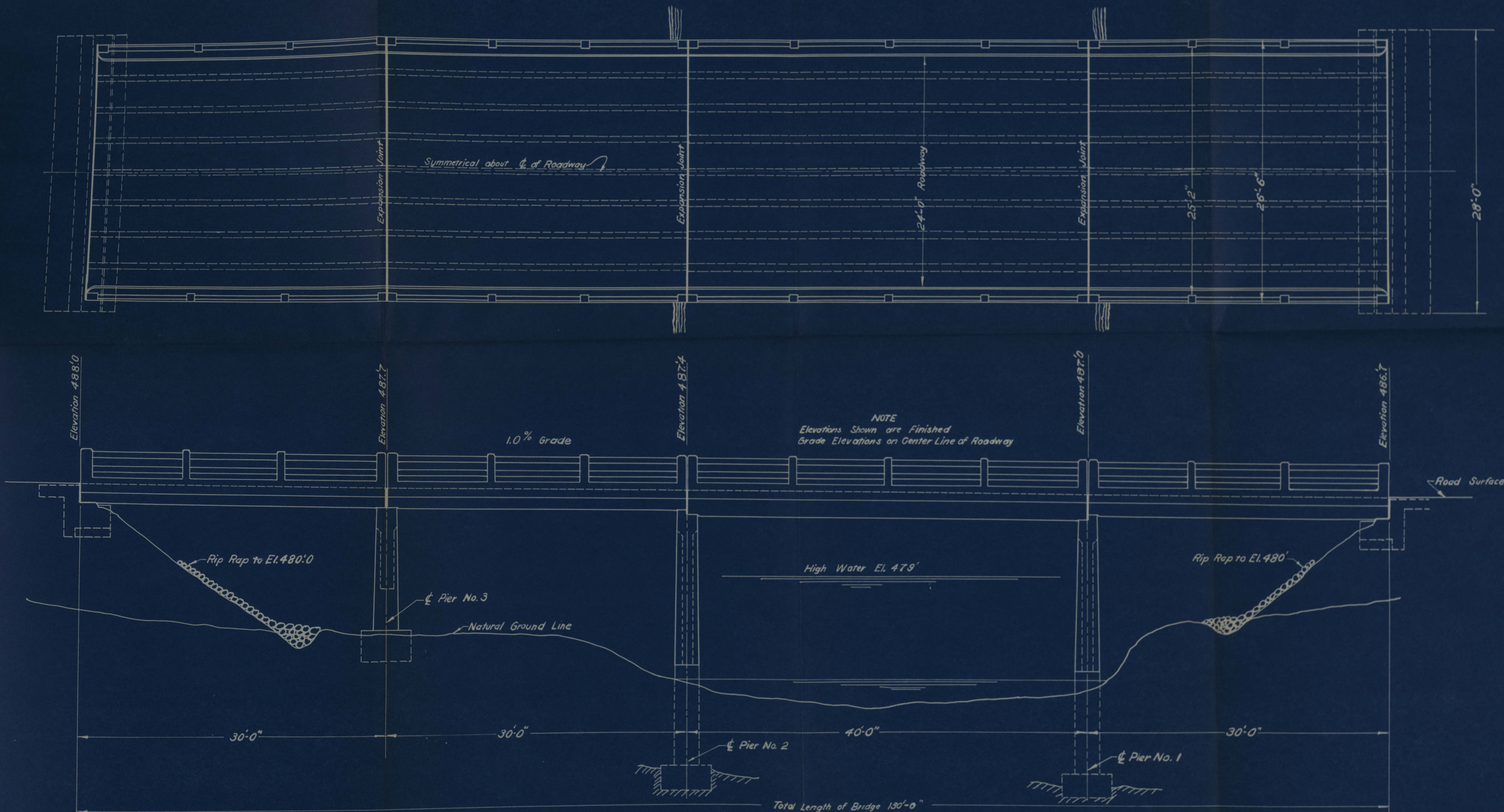


FIGURE 4 ENCASED-I BEAM DESIGN
Scale $\frac{3}{16}$" = 1'-0"

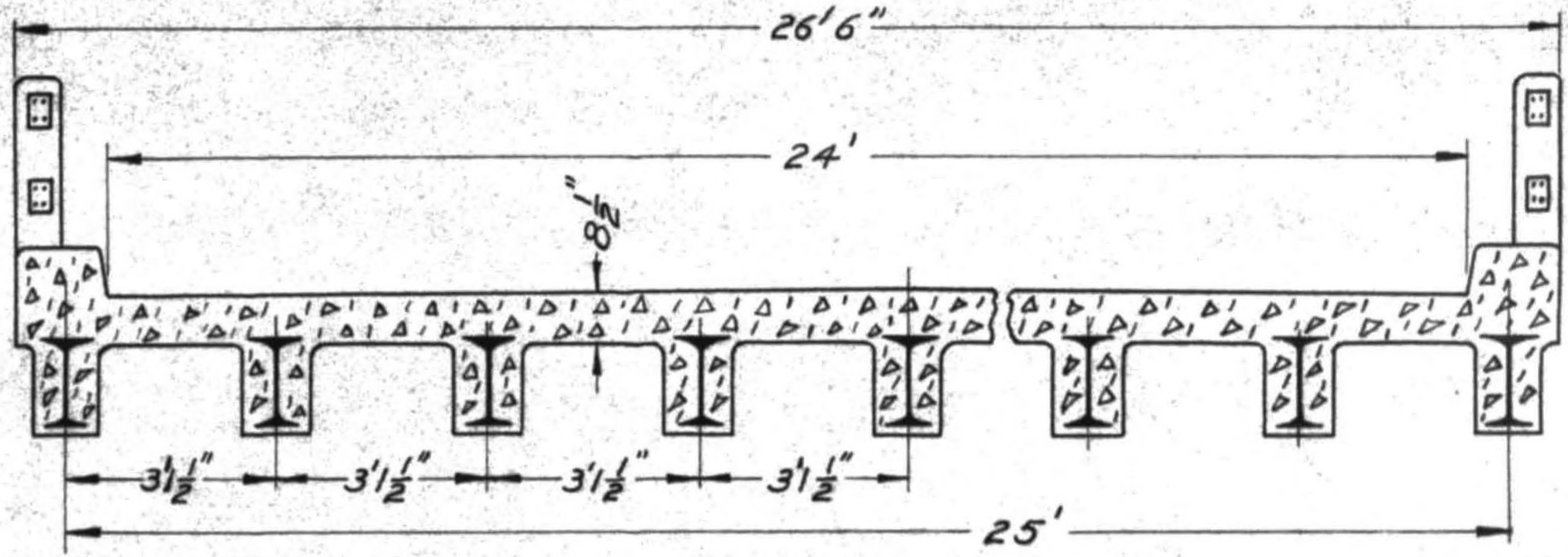


FIGURE 5

CROSS SECTION OF THE ROADWAY

SCALE:- $\frac{3}{8}$" = 1FT

Allowable Working Stresses

f_s = Allowable unit tensile stress in steel = 18,000 lb/in2

f'_c = Ultimate crushing strength of concrete = 3,000 lb/in^2

f_c = Allowable compressive stress in concrete = .4 f'_c = 1,200 lb/in^2

except over supports of continuous beams = .45 f'_c = 1,350 lb/in2

$n = \frac{E_s}{E_c} = 10$

v = Allowable unit shearing stress

value of v	web reinforcing required	special anchorage of longitudinal steel required
less than .02 f'_c	NO	No
between .02 f'_c and .03 f'_c	No	Yes
" .03 f'_c " .06 f'_c	Yes	No
" .06 f'_c " .12 f'_c	Yes	Yes

v_c = Shear concrete will carry

u = Allowable bond stress

for plain bars u = .04 f'_c = 120 lb/in^2 or less

for deformed bars = u = .05 f'_c 150 lb/in^2 or less

ΣO = Total perimeter of tension bars required to carry bond.

Slab Design for Encased "I" Beam Bridge

Design of concrete slab for the 30 foot span.

The slab is supported by nine encased steel "I" beams spaced $3\frac{1}{8}$ feet center to center, and is considered as a beam continuous over nine supports and free at both ends.

To locate the position of the truck wheel on the outside span which will produce a maximum positive moment in the slab between two "I" beams a load P = 1# will be placed in several positions in between the two supports and the negative moment over the second support found by the moment distribution method. (The opposite wheel falling on the alternate span produces a moment practically negligible but on the side of safety. Therefore only one wheel was considered.)

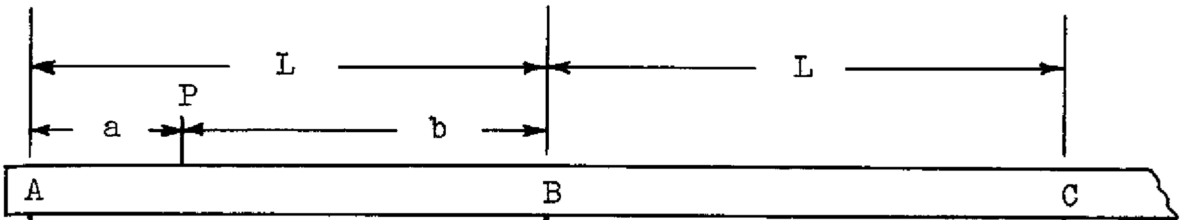
Let a = distance from the load to the free support
Let b = distance from the load to the second support

Since the slab is of uniform thickness the stiffness factor is 1 for all spans.

Slab design for Encased "I" Beam cont.

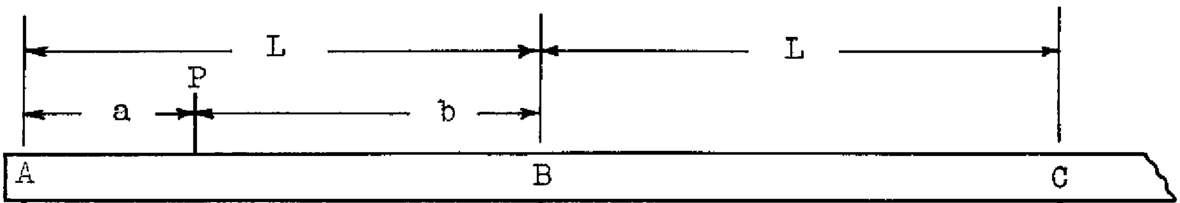
$M_{FAB} = Pa(\frac{b}{L})^2$ when $a = \frac{L}{4}$ $b = \frac{3L}{4}$ $M_{FAB} = \frac{9}{64}PL$

$M_{FBA} = Pb(\frac{a}{L})^2$ $M = \frac{3}{64}PL$



A	B	B	C	C
+.140	-.047	0.	0.	0.
-.140	+.0235	+.0235	0.	0.
+.01175	-.070	0.	+.01175	
-.01175	+.035	+.035	-.0058	-.0058
+.0175	-.00588	-.00588	+.0175	
-.0175	+.00588	+.00588		
0.	-.0585	+.0585		

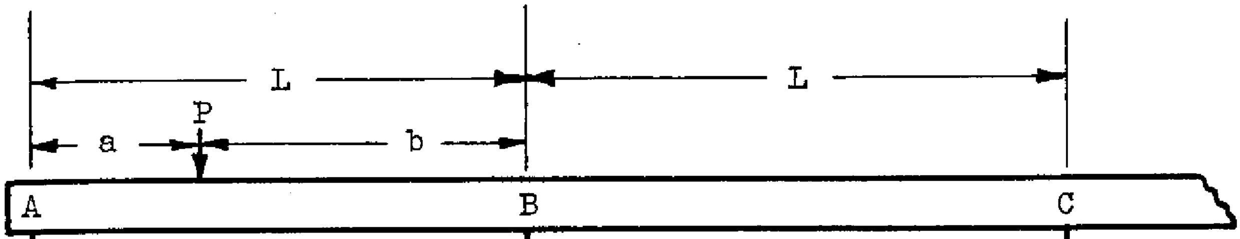
when $a = \frac{3}{8}L$ $b = \frac{5}{8}L$ $M_{FAB} = .147PL$ $M_{FBA} = .088PL$



A	B	B	C	C
+.147	-.088	0.	0.	0.
-.147	+.044	+.044	0.	0.
+.022	-.0735	0.	+.022	
-.022	+.03675	+.03675	-.011	-.011
+.01838	-.011	-.0055	+.01838	
-.01838	+.00825	+.00825		
0.	-.0835	+.0835		

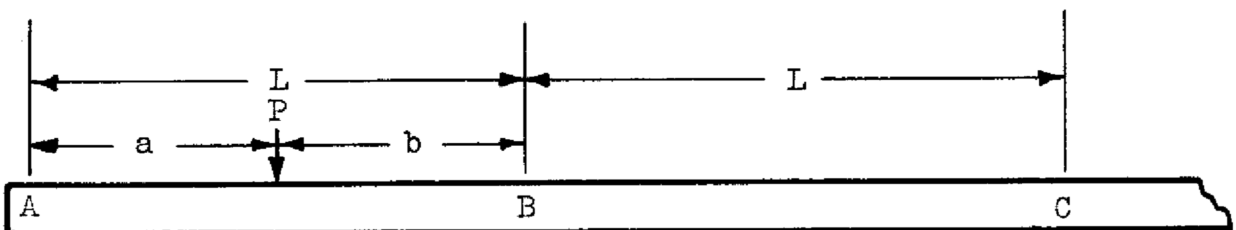
Slab Design for Encased "I" Beam cont.

when $a = \frac{7}{16}L$ $b = \frac{9}{16}L$ $M_{FAB} = .139PL$ $M_{FBA} = .$ PL



+.139	-.108	0.	0.	0.
-.139	+.054	+.054	0.	0.
+.027	-.0695	0.00	+.027	0.
-.027	+.03475	+.03475	-.0135	-.0135
+.01738	-.0135	-.00675	+.03475	
-.01738	+.01013	+.01013		
0.	-.0921	+.0921		

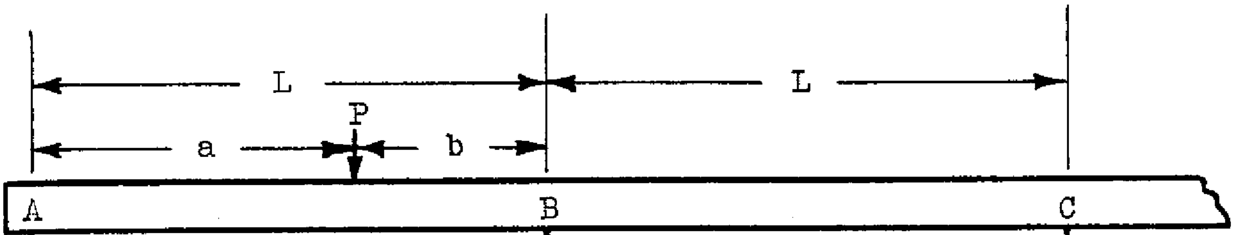
when $a = \frac{L}{2}$ $b = \frac{L}{2}$ $M_{FAB} = .125$ $M_{FBA} = .125$



+.125	-.125	0.	0.	0.
-.125	+.0625	+.0625	0.	0.
+.03125	-.0625	0.	+.03125	0.
-.03125	+.03125	+.03125	-.01563	-.0156
+.01563	-.01563	-.00781	+.01563	0.
-.01563	+.01172	+.01172		
0.	-.0977	+.0977		

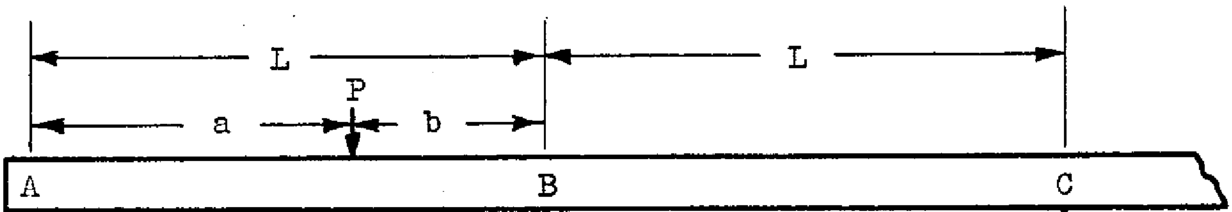
Slab Design for Encased "I" Beam cont.

when $a = \frac{5L}{8}$ $b = \frac{3L}{8}$ $M_{FAB} = .088PL$ $M_{FBA} = .147PL$



A	B (left)	B (right)	C (left)	C (right)
+.088	-.147	0.	0.	0.
-.088	+.0735	+.0735	0.	0.
+.03675	-.044	0.	+.03675	0.
-.03675	+.022	+.022	-.01838	+.01838
+.011	-.01838	-.00919		
-.011	+.01378	+.01378		
0.	-.100	+.100		

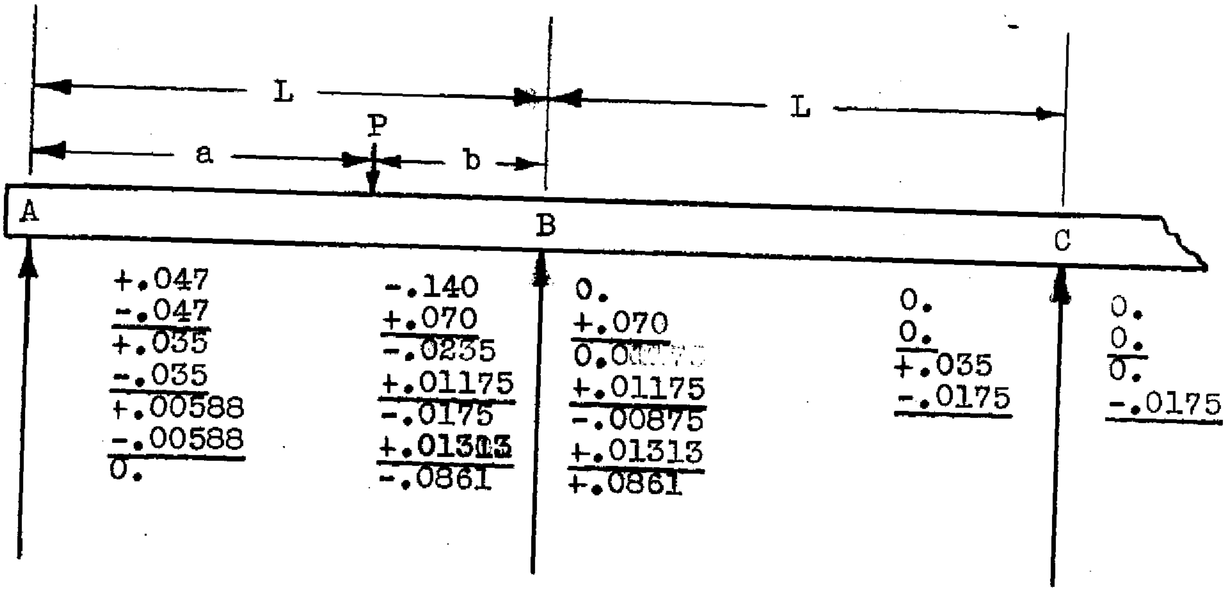
when $a = \frac{11}{16}L$ $b = \frac{5L}{16}$ $M_{FAB} = .0672PL$ $M_{FBA} = .148PL$



A	B (left)	B (right)	C (left)	C (right)
+.0672	-.148	0.	0.	0.
-.0672	+.074	+.074	0.	0.
+.037	-.0336	0.	+.037	0.
-.037	+.0168	+.0168	-.0185	-.0185
+.0084	-.0185	-.00925	+.0084	0.
-.0084	+.01388	+.01388		
0.	-.0954	+.0954		

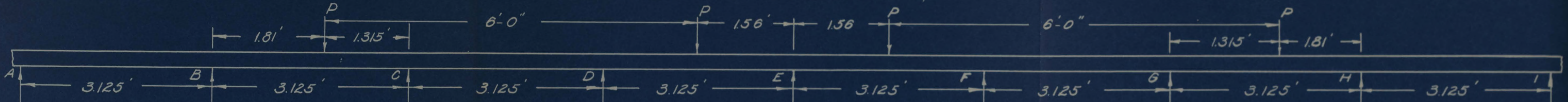
Slab Design for Encased "I" Beam cont.

when $a = \frac{3}{4}L$ $b = \frac{1}{4}L$ $M_{FAB} = .047PL$ $M_{FBA} = .140PL$



CROSS SECTION OF THE ROADWAY

STIFFNESS FACTORS FOR ALL SPANS = 1



A	B	B	C	C	D	D	E	E	F	F	G	G	H	H	I
0	0	+.321	-.442	0	0	+.392	-.392	+.392	-.392	0	0	+.442	-.321	0	0
0	-.1605	-.1605	+.221	+.221	-.196	-.196	0	0	+.196	+.196	-.221	-.221	+.1605	+.1605	0
-.08025	0	+.1105	-.08025	-.098	+.1105	0	-.098	+.098	0	-.1105	+.098	+.08025	-.1105	0	+.08025
+.08025	-.0552	-.0552	+.08912	+.08912	-.0522	-.0552	0	0	+.0552	+.0552	-.08912	-.08912	+.0552	+.0552	-.08025
-.0276	+.04012	+.04456	-.0276	-.0276	+.04456	0	-.0276	+.0276	0	-.04456	+.0276	+.0276	-.04456	-.04012	+.0276
+.0276	-.04234	-.04234	+.0276	+.0276	-.02228	+.02228	0	0	+.02228	+.02228	-.0276	-.0276	+.04234	+.04234	-.0276
0	-.2179	+.2179	-.212	+.212	-.118	+.118	-.517	+.517	-.118	+.118	-.212	+.212	-.217	+.217	0

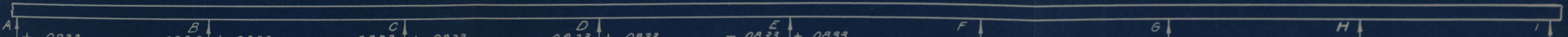
FOR MAXIMUM NEGATIVE MOMENT THE TRUCK WHEELS ARE PLACED IN THE MIDDLE OF SPANS D-E AND E-F AS SHOWN

$M_{FBC} = M_{FHG} = P\,1.81\left(\frac{1.35}{3.125}\right)^2 = .321$ $\quad M_{FCB} = M_{FGH} = P\,1.315\left(\frac{1.81}{3.125}\right)^2 = .442$ $\quad M_{FDE} = M_{FED} = M_{FEF} = M_{FFE} = P\frac{L}{8} = .392$

MAXIMUM NEGATIVE MOMENT DUE TO L.L. OCCURS OVER SUPPORT E AND ITS MAGNITUDE IS .517 P

TO DETERMINE THE NEGATIVE MOMENT DUE TO D L LET W = WEIGHT PER FT. OF SPAN

$M_{FAB} = M_{FBA} = M_{FBC} = ETC. = \frac{WL}{12} = .0833W$ FOR UNIT SPAN



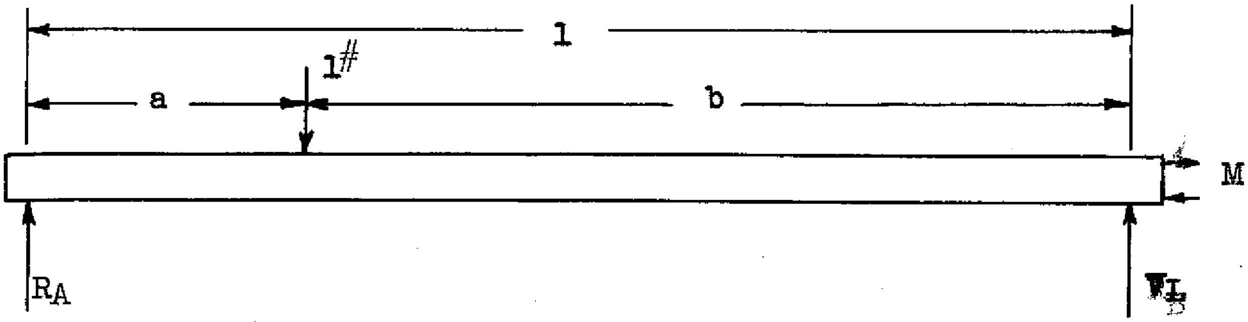
A	B	B	C	C	D	D	E	E
+.0833	-.0833	+.0833	-.0833	+.0833	-.0833	+.0833	-.0833	+.0833
-.0833	0	0	0	0	0	0	0	0
0	-.04165	0	0	0	0	0	0	0
0	+.02082	+.02082	0	0	0	0	0	0
+.01041	0	0	+.01041	0	0	0	0	0
+.01041	0	0	-.00520	-.00520	0	0	0	0
0	-.104	+.104	-.078	-.078	-.0833	+.0833	-.0833	+.0833

Slab Design for Encased "I" Beam cont.

when $a = .25$ $\quad M = .0585\ P$ $\quad \Sigma M_{VL} = R_A \times 1 + M - 1 \times b = 0$

$R_A = .75 - .0585 = .6915$ $\quad$ Hence Moment at section .25 from R_A

$M_{.25} = R_A \times a$ $\quad M_{.25} = .6915 \times .25$ $\quad M_{.25} = .1725$



value of a	value of b	value of M	value of R_A	value of Moment under the load
.25	.75	.0585	.6915	.1728 5
.375	.625	.0835	.5415	.203
.438	.562	.0921	.4699	.2055
.5	.5	.0977	.4023	.201
.625	.375	.1	.275	.1717
.688	.312	.0954	.2166	.149
.75	.25	.0861	.1639	.1228

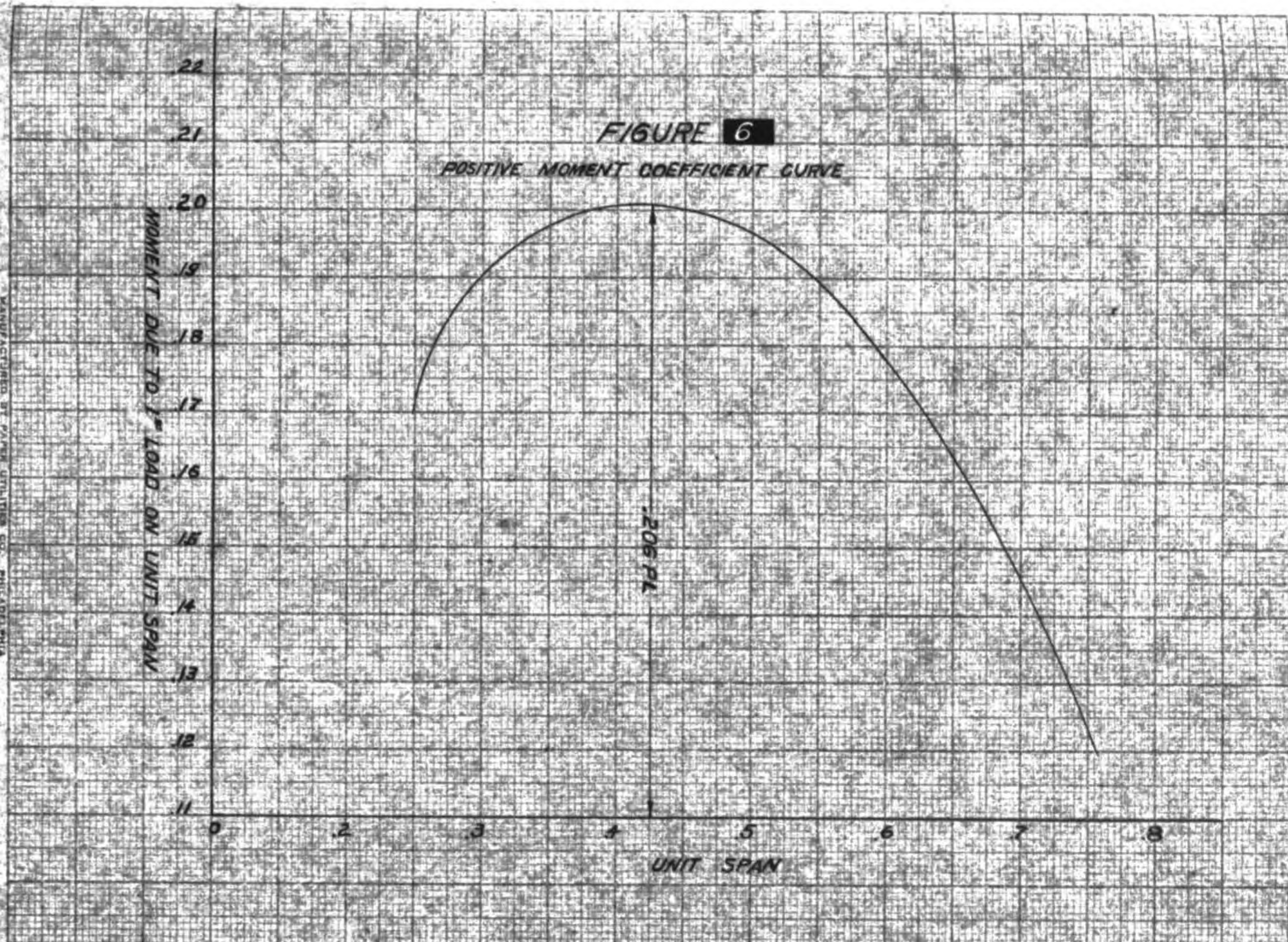
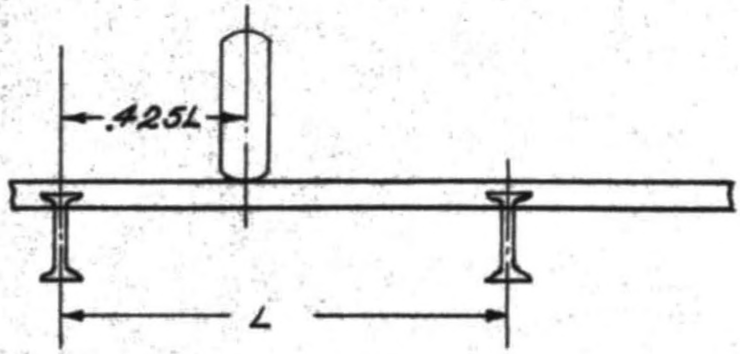


FIGURE 6
POSITIVE MOMENT COEFFICIENT CURVE
MOMENT DUE TO 1# LOAD ON UNIT SPAN
UNIT SPAN
.206 PL
.22
.21
.20
.19
.18
.17
.16
.15
.14
.13
.12
.11
0
.2
.3
.4
.5
.6
.7
.8
MANUFACTURED BY PAPER UTILITIES CO. PHILADELPHIA
20 LINES = 1 INCH

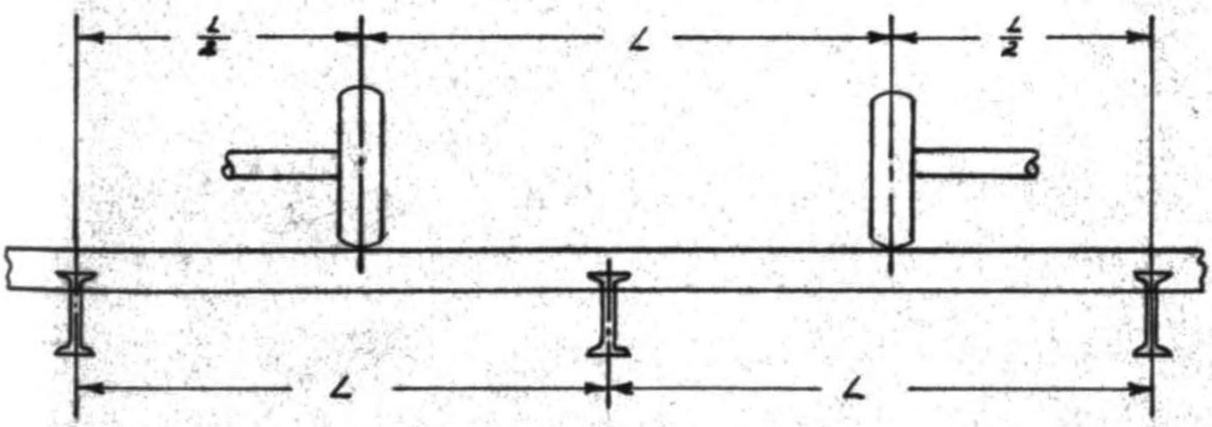
Figure No. 6 on page 9 shows that the maximum positive moment due to LL occurs when the wheel load is about .425 L. from the free end as shown.



The magnitude of this moment (from fig. 6) is about .206 PL.

where: P = wheel load
L = distance between supports

The maximum negative moment due to LL occurs over the support when the wheel loads are placed as shown



The magnitude of the negative moment due to LL (from page 17) is .517 PL.

Slab Design for Encased "I" Beam cont.

Assuming the width of the concrete encasement of the beam = 1 ft. Let S = clear span of the slab. Then $S = 3.125 - 1$ or $S = 2.125$. Let E_m = the effective width for moment $= \frac{2}{3} S$ $E_m = \frac{2 \times 2.125}{3} = 1.42$ ft.

Let E_n = effective width for shear $= 3.33\ d$ where d = Effective depth of slab assuming d = 7 in. E for shear $= \frac{3.33 \times 7}{12}$ 2.01 ft.

then the thickness of slab $t = 7 + 1.5 = 8.5$ in.

UDL = weight of slab + covering $= 8.5 \times 12.5 + 14 = 120$ lb/ft^2

Rear wheel load on the slab for moment $= \frac{\text{wheel load}}{\text{eff. width for moment}}$

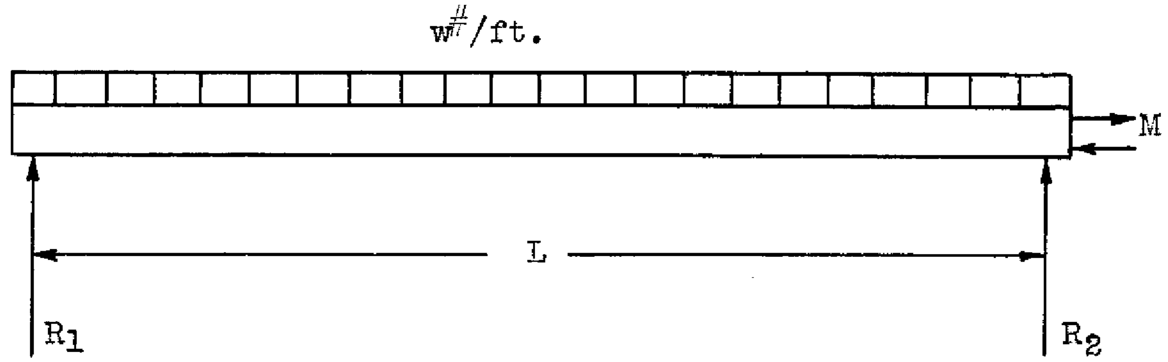
" " " " " " " " $= \frac{12000}{1.42 \times 1000} = 8.45$ Kips

" " " " " " " **Shear** $= \frac{\text{wheel load}}{\text{eff. width for shear}}$

" " " " " " " " $= \frac{12,000}{2.01 \times 1000} = 5.97$ Kips

For the positive UDL moment, showing the portion between the first two supports as freebody and taking moments about V_{LR2} Value of M is obtained from moment distribution page **17** $M = .104\ WL = .104\ wL^2$

$$\Sigma M_{VLR_2} = R_1 L + .104\ wL^2 - \frac{wL^2}{2} = 0 \qquad R_1 = .396\ wL$$



Slab design for Encased "I" Beam cont.

The positive moment is maximum when shear = 0. Writing shear equation and equating it to 0 and solving for distance to the point of maximum postive moment

$V = .396 - wx = 0 \qquad x = .396\ L = .396 \times 2.125 = .842'$

Maximum positive moment is under the LL when the wheel is = .425 L or .906 feet from the end support. (see page 19)

Total (+) M = DLM + LLM + ILM

$DLM = R_1x - \frac{wx^2}{2} = .396 \times 120 \times 2.125 \times .8442 - \frac{120}{2}(.842)^2 =$ 42 ft. lb.

+ L.L.M. = .2077 PL = 2077 x 8.45 x 1000 x 2.125 = 3730'#

+ I.L.M. = 30% of L.L. M. = 3730 x.3 = 1120'#

Total M = 42 + 3730 + 1120 = 4892'# use 4900'#

Maximum negative moment is over the center support R_E =

$-DLM$ over R_E = $.0833\ WL = .0833\ wL^2 = .0833 \times 120 \times (2.125)^2 =$ -45'#

-LLM = .517 P = .517 x 8.45 x 1000 = -4370'#

-ILM = CLLM x .3 = -1315'#

- Total Moment = 5730'#

Shear	Loading	At cent.	At supp.	At PI
"	DL	0	$\frac{120 \times 2.125}{2}$ = 128	
"	CLL	$\frac{5970}{2}$ = 2985	5970	$\frac{2985 + 6098}{2}$
"	Total	" 2985 #	6098#	4541 #

Slab Design for Encased "I" Beam cont.

$bd^2 = \frac{M}{K}$ Where M = moment in inch pounds

K from table #3 for $n = 10$, $f_c = 1200\#/in^2$, $f_s = 18000\#/in^2$ $K = 208.3$

$d = \sqrt{\frac{5730 \times 12}{208.3 \times 12}}$ $d = 5.24$ in.

$d = \frac{V}{bJv}$ or $d = \frac{6098}{12 \times .87 \times 90}$ or $d = 6.48$ in.

use 7 in. for d

A_s top $= \frac{M}{f_s Jd} = \frac{5730 \times 12}{18000 \times .87 \times 7}$ $A_s = .627\ in^2$

A_s bottom $= \frac{M}{f_s Jd} = \frac{4900 \times 12}{18000 \times .87 \times 7}$ $A_s = .537\ in^2$

$\Sigma 0 = \frac{V}{UJd}$ at top $\Sigma 0 = \frac{6098}{150 \times .87 \times 7}$ $\Sigma 0 = 6.66$ in.

at PI $\Sigma 0 = \frac{4541}{150 \times .87 \times 7}$ $\Sigma 0 = 5.01$ in.

at bottom $\Sigma 0 = \frac{2985}{150 \times .87 \times 7}$ $\Sigma 0 = 3.26$ in.

Slab Design for Encased "I" Beam cont.

Since v used = .03 $f_c' = 90\#/in^2$ Special anchorage of longitudinal steel is required, or seven twelfths of the steel at the bottom must be carried through,

$\frac{7}{12} \times .537 = .31\ in^2$ of the steel at the bottom.

Bottom use 5/8 Round bar 12" C to C. Straight through MK-S-500

" use $\frac{1}{2}$ Square bar 12" C to C bent up over the supports MK-S-450

Top use $\frac{3}{4}$ Round 12" C to C straight through MK-S-601

" " $\frac{3}{4}$ Round 12" C to C beyond P.I. and hooked MK-S-600

Total A_s in bottom used = .56 in^2 $\Sigma 0$ = 3.96 in.

" " " top " = 1.13 in^2 $\Sigma 0$ = 6.72 in.

At P.I. $\Sigma 0$ = 7.32 in.

For longitudinal reinforcement use 3/8 ø bars 12 in Cto C for full length of span MK-S-300

Use $\frac{1}{2}$ ø stirrups 12 C to C MK-SB-200(see page 36 fig.8)

Railing is designed to resist a horizontal force of 150# per foot and a vertical force of 100# per foot. Moment in railing is greatest at the center.

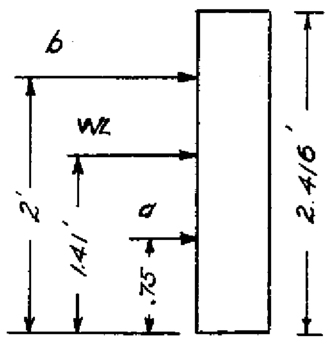
Moment in railing due to horizontal load

$$M = \frac{wL^2}{8} \text{ or } M = \frac{150(9.3)^2}{8} \text{ or } M = 1630'\#$$

Moment in railing due to vertical load

$$M = \frac{100 \times 9.3}{8} \text{ or } M = 1080'\#$$

Forces acting on the vertical post are the reactions from the two railings. (force a and b) + the force acting on the post wL



a = b = 150 x 8.3 = 1240#

Moment on post = 1240 x .75 = 1240 x 2 = 3415'#

A_s in vertical post $= \frac{M}{f_s J d} = \frac{3415 \times 12}{18000 \times .87 \times 6.5} = .40 \text{ in}^2$

Use 2-½ round bars 5" C to C bent as shown in fig. 8 page 36

MK-P-400

A_s in railing $= \frac{M}{f_s J d} = \frac{1630 \times 12}{18000 \times .87 \times 3} = .41 \text{ in}^2$

Use 4-½ square bars spaced 2" C to C in each horizontal plane and 3" C to C in vertical plane MK-R-400

Design of curb

The curb is designed to resist a lateral force of 500# per lineral foot of curb, applied at the top of the curb. Assuming the curb to be a cantilever beam

$$M = 500 \times 9 = 4500 \text{ in. lbs./ft. of length}$$

$$A_s = \frac{M}{f_s jd} = \frac{4500}{18000 \times .87 \times 13} = .022 \text{ in}^2\text{/ft. of width}$$

Use $\frac{1}{2}\phi$ bars in vertical plane 12 C to C bent as shown in fig. 8 Page 36 MK-C-400

Use 6 5/8 round bars for the full length of span as shown in fig. 8 Page 36 MK-C-500

Design of 30' span encased I beams

Using 9 I beams, as shown in fig. 5 page 10 the spacing of beams $S = \frac{L}{8}$ where L is distance from center to center of external beams. $S = \frac{25}{8} = 3.125'$ The wheel concentration, or portion of wheel load carried by one beam $=$

$$\frac{S}{4.5} = \frac{3.125}{4.5} = .694$$

Then load of rear wheel on a beam $= 12000 \times .694 = 8330\#$

" " " front " " " " $= 3000 \times .694 = 2080\#$

The center of gravity of a 15 ton truck is 2.8 feet from the center line of rear axle. Maximum moment in the beam will occur when the rear wheel is as far from one end of the span as the center of gravity of the truck is from the other. (as shown in fig. 7 page 28)

The maximum moment will occur under the wheel nearest the center of gravity.

$\Sigma M_{R_2} = 15000 \times 13.6 - 30 R_1 = 0$ $\quad\quad R_1 = 6800\#$

L.L.M. $= (6800 \times 13.6)$(wheel concentration)

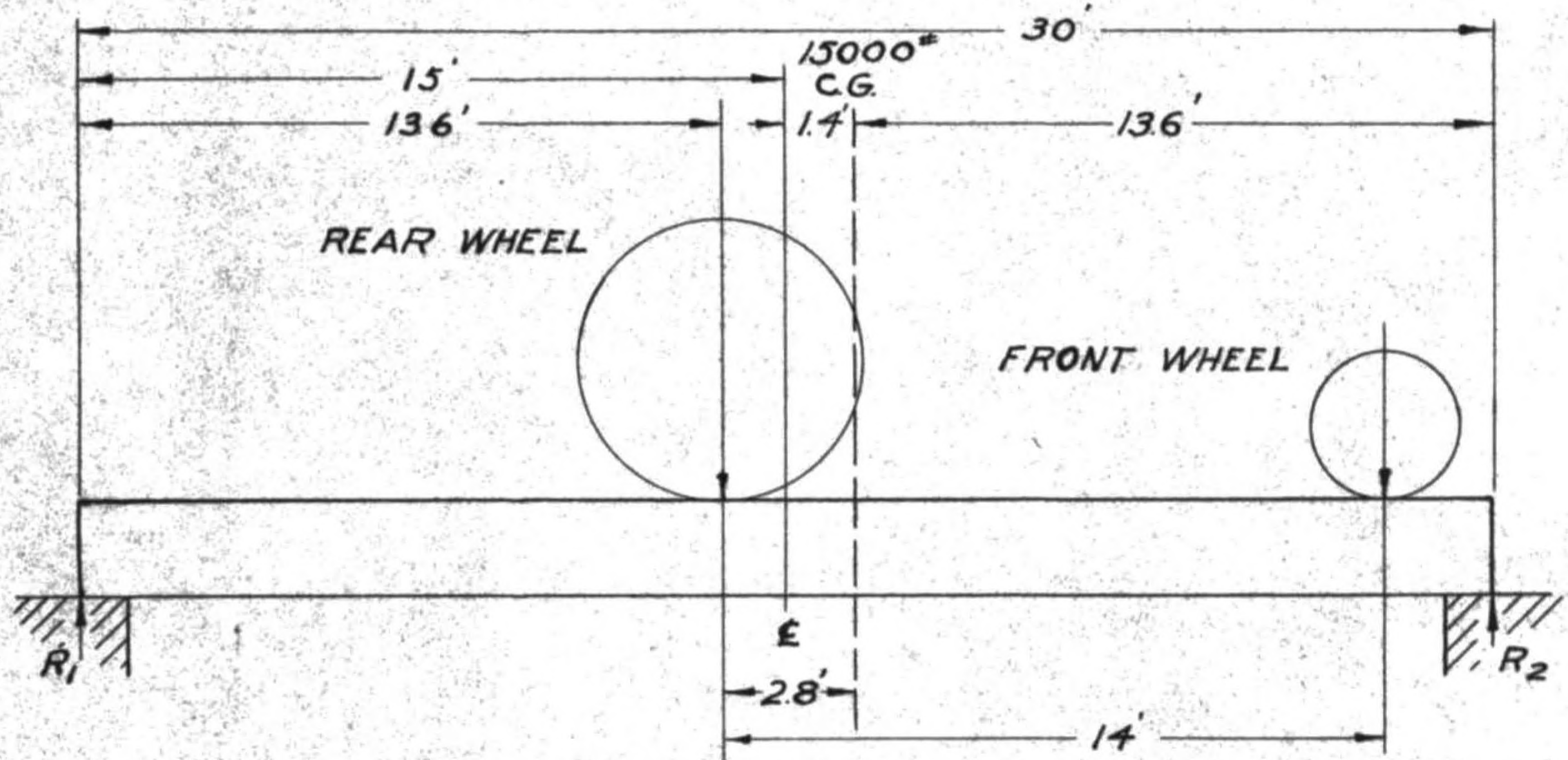
" $= 6800 \times 13.6 \times .694 = 64200\#'$

Imp.M $= 64200 \times .3 = 19250\#'$

Assuming a 16" 36# Carnegie beam and encasement 12" wide by 16" deep.

Weight of encasement $= \dfrac{12 \times 16 - \frac{3}{4}\text{ area of I beam section}}{144} \times 150$

" " " $= 192\#$



POSITION OF TRUCK WHEELS FOR MAXIMUM MOMENT AT THE CENTER OF SPAN

FIGURE 7

SCALE: $\frac{1}{4}$" = 1 FT.

Design of 30' span I beam cont.

Total DL per foot of span on the beam = wt. of slab per ft^2 x span + wt. of I beam per ft. + wt. of encasement per foot.

Total DL per ft. of length = 120 x 3.125 + 36 + 192

" " " " " " = 603#

$$D.L.M. = \frac{wLx}{2} - \frac{wx^2}{2} \quad \text{where } x = 13.6'$$

$$" = \frac{603 \times 30 \times 13.6}{2} - \frac{603 \times (13.6)^2}{2} = 67500\#'$$

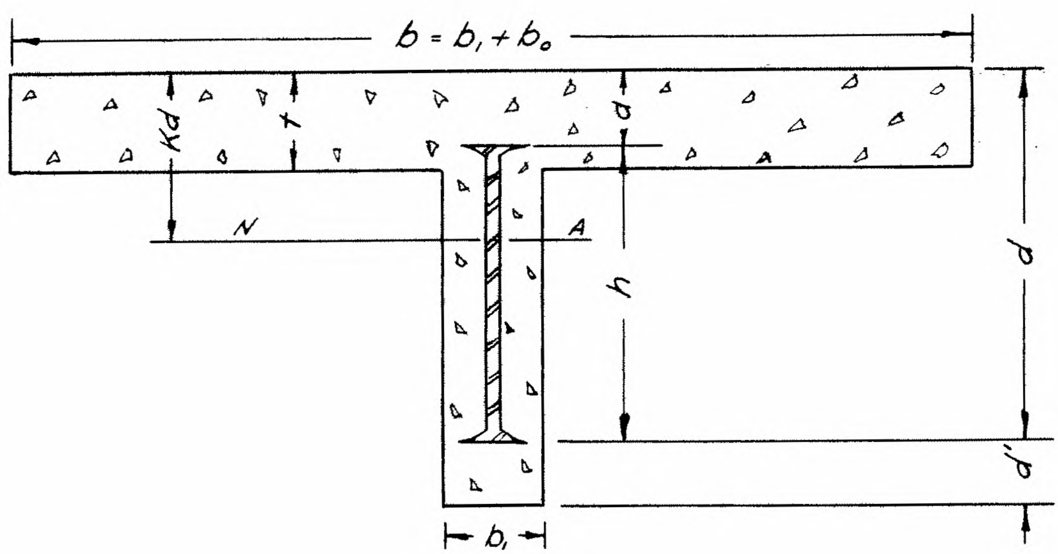
Total moment on the beam = C.L.M. + Im.M + D.L.M.

" " " " " = 64200#' + 19250#' + 67500#'

" " " " " = 150950#' or 1,810,000#"

Let A_s = area of cross-section of I beam

" I = moment of inertia about N-A of I beam



A_s = 10.59 in^2. I = 446.3in^4. b_1 = 12 in.

t = 8.5 in. a = 7.19 in. d = 23.18 in.

f'_c = 3,000#/in^2. f_c = 1200#/in^2. h = 16"

n = 10 k = .4

Design of 30' span I beam cont.

"T" beam dimensions. Flange forming part of a floor system.

(1) b - not more than one fourth of the span length of beam

(2) b - not more than the distance center to center of beam

(3) b - not more than six times the width of beam.

(4) b - not more than eight times the least thickness of the slab plus the width of the stem.

(1) $b = \frac{30}{4} = 7.5'$

(2) $b = 3.125'$

(3) $b = 6 \times 1 = 6'$

(4) $b = 8 \times \frac{8.5}{12} + 1 = 6.57'$

The smallest b will be used which is 3.125' or b 37.5"

$$b_o = \frac{(b - b_1)(kd)^3}{3t\left\{\frac{t^2}{12} + (kd - \frac{t}{2})^2\right\}} = \frac{(37.5 - 12)(.4 \times 23.18)^3}{3 \times 8.5\left[\frac{(8.5)^2}{12} + (.4 \times 23.18 - \frac{8.5}{2})^2\right]}$$

$b_o = 25.5$ $b = b_1 + b_o$ $b = 12 + 25.5 = 37.5$

I = the moment of inertia of the section

I_s = the moment of inertia of the I beam

A_s = the cross-section area of I beam

$$I = I_s + A_s(\frac{h}{2} + a - kd)^2 + \frac{b(kd)^3}{3h}$$

$$I = 446.3 + 10.59(\frac{16}{2} + 7.19 - .4 \times 23.18)^2 + \frac{37.5(.4 \times 23.18)^3}{3 \times 10}$$

$I = 1812$ in.^{4}

$M = \frac{nf_cI}{kd}$ where M is the total resisting moment of the section.

$M = \frac{10 \times 1200 \times 1812}{.4 \times 23.18} = 2,340,000$ in. lb. OK since it is the lightest section to carry the moment.

V is greatest when the rear wheel is at the support.

V due to wheel load = 8330#

V " " impact " = 8330 x .3 = 2500#

V " " D.L. $= \frac{wL}{2} = \frac{603 \times 30}{2} = 9045\#$

Total V = 8330 + 2500 + 9045 = 19,875#

Check for shearing strees which must not exceed .02 f'_c or not exceed .02 x 3000 or must not exceed 60#/in^2

$$v = \frac{aV}{nI}(kd - \frac{a}{2}) = \frac{7.18 \times 19875}{10 \times 1812}(.4 \times 23.18 - \frac{7.18}{2}) = 44.6\#/in^2$$

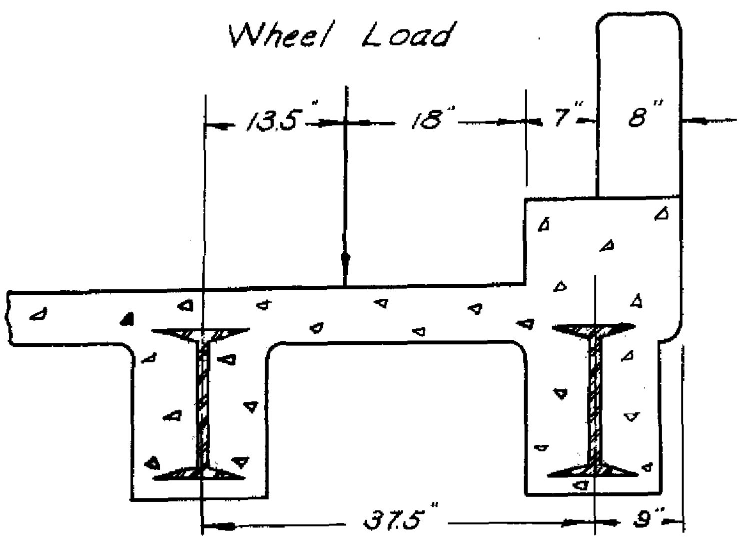
Check for bearing area between beam and girder

Required area $= \frac{V}{600} = \frac{19875}{600} = 33.1\ in^2$

Area available = width of flange x width of support

" " = 7 x 10 = 70 in^2

External Beam



The center of the wheel can not come nearer than 2 ft. to the center line of the exterior beam; so the moment and shear for the exterior beam due to C.L.L. are $(\frac{3.18 - 2.08}{3.18})$ time the values for the interior beam. D.L. on exterior beam equals the sum of weights of railing posts, railings, curb, slab and covering.

wt. of railing posts $= \frac{2}{3} \times \frac{2}{3} \times \frac{9}{4} \times 6 \times 150 = 900^{\#}$

wt. of railing $= \frac{1}{2} \times \frac{2}{3} \times 3.42 \times 7 \times 2 \times 150 = 2390^{\#}$

wt. of curb $= 1.208 \times .75 \times 30 \times 150 = 4080^{\#}$

wt. of slab $= 2.25 \times .708 \times 30 \times 150 = 7170^{\#}$

wt. of covering $= 2.6 \times 30 \times 15 = \underline{1170^{\#}}$

Total weight $= 15710^{\#}$

Weight per foot of span $= \frac{15710}{30} = 524^{\#}$

$V = 7850^{\#}$

$$\text{D.L.M.} = \frac{524 \times (30)^2}{8} = 59{,}000^{\#'}$$

$$\text{CLLM} = \frac{3.12 - 2.08}{3.12}(64200) = 21{,}800^{\#'}$$

$$\text{Imp. M} = 23800 \times .3 = 7{,}140^{\#'}$$

Total moment on exterior beam = $87{,}940^{\#'}$ or 1,028,000in-lb. Moment is less than on interior beam. Same size beam was used as the interior beam.

Design of the 40' Span Encased I Beams

Use the same spacing of I beams and the same slab as in the 30' span. Since the truck preceding and the truck following the one on the span are 30' from the front and rear axle respectively, it is obvious that the greatest moment will occur when only one truck is on the span and in the same position relative to the center line of the span as in the 30' span, shown in figure 7 page 28

$$\Sigma M_{R_2} = 15000 \times 18.6 - 40R_2 = 0 \qquad R_2 = 9300\#$$

$$\text{L.L.M.} = 9300 \times 18.6 \times .694 = 120000\#'$$

$$\text{I.M.} = 120000 \times .3 = 36000\#'$$

Assume 21 in. 59 lb. C. I beam $A_s = 17.36$

Wt. of encasement approx. $\left(\frac{12 \times 19 - 11}{144}\right) 150 = 251\#/\text{ft.}$

D.L. per foot = wt. of slab + wt. of beam + wt. of encasement

" " " = 120 x 3.125 + 59 + 251 = 685# per foot of span

$\text{D.L.M.} = \frac{wL}{2}x - \frac{wx^2}{2}$ where $x = 18.6'$

$$\text{D.L.M.} = \frac{685 \times 40}{2} \times 18.6 - \frac{685\ (18.6)^2}{2} = 137000\#'$$

Total M = L.L.M + I.M. + D.L.M.

" " = 120,000 + 36,000 + 137,000

" " = 293,000#' or 3,520,000#"

$A_s = 17.36$ in^2 $\quad a = 7.18$ in. $\quad n = 10 \quad d = 28.18$ in.

$I_s = 1246.8$ in^4 $\quad f_c = 1200\#/\text{in}^2 \quad k = .4 \quad b_1 = 12$ in.

$b' = 12$ in. $\quad t = 8.5$ in. $\quad f_s = 18000\#/\text{in}^2 \quad b = 37.5$

Design of 40' span I beam cont.

$$b_o = \frac{(b - b_1)(kd)^3}{3t\left\{\frac{t^2}{12} + (kd - \frac{t}{2})^2\right\}} = \frac{(37.5 - 12)(.4 \times 28.18)^3}{3 \times 8.5\left\{\frac{(8.5)^2}{12} + (.4 \times 28.18 - \frac{8.5}{2})^2\right\}}$$

b_o = 25.8 but b can not be more than 37.5 in. Hence use

b = 37.5 in.

$$I = I_s + A_s(\frac{h}{2} + a - kd)^2 + \frac{b(kd)^3}{3n}$$

$$I = 1246.8 + 17.36(\frac{21}{2} + 7.18 - .4 \times 28.18)^2 + \frac{37.5(.4 \times 28.18)^3}{3 \times 10}$$

$I = 3745.0 \text{ in}^4$

$$M \text{ of the section} = \frac{n\, f_c\, I}{kd} = \frac{10 \times 1200 \times 2494.4}{.4 \times 25.18}$$

$M = 398000^{\#'}$ or $4,780,000^{\#''}$ for total resisting moment of the section. This size I beam will be used since it is the lightest beam that will carry the moment.

V is greatest when the wheel is at the support

V due to wheel load = $9300^{\#}$

V " " IL " = $9300 \times .3 = 2790^{\#}$

V " " DL " $= \frac{685 \times 40}{2} = 13700^{\#}$

Total V = 9300 + 2790 + 13700 = $25790^{\#}$

$$v = \frac{aV}{nI}(kd - \frac{a}{2}) = \frac{7.18 \times 25790}{10 \times 3745}(.4 \times 28.18 - \frac{7.18}{2}) = 37.9$$

less than $.02\, f'_c$ or less than 60 OK

Check for bearing area between beam and girder

$$\text{Required area} = \frac{V}{600} = \frac{25790}{600} = 43 \text{ in}^2$$

Available area = width of flange x width of support

" " $= 8.23 \times 11 = 90.7 \text{ in}^2$

Design of 40' span I beam cont.

Check for exterior beam size

DL on exterior beam = wt. of railing + wt. of curb + wt. of slab + portion of LL

Railing posts $= \frac{2}{3} \times \frac{2}{3} \times 2.5 \times 6 \times 150 = 1000^{\#}$

Railing $= \frac{1}{2} \times \frac{1}{3} \times 3.3 \times 18 \times 150 = 1486^{\#}$

Curb $= 1.3 \times .75 \times 40 \times 150 = 5850^{\#}$

Slab $= 2.31 \times .708 \times 40 \times 150 = 9800^{\#}$

Covering $= 1.6 \times 15 \times 40 = 960^{\#}$

Total D.L. $= 19096^{\#}$

M due to D.L. $= \frac{W}{2}x - \frac{Wx^2}{L2}$ where $x = 18.6$ '

" " " " $= \frac{19096 \times 18.6}{2} - \frac{19096(18.6)^2}{40 \times 2}$

" " " " $= 99100^{\#'}$ or $1,199,000^{\#''}$

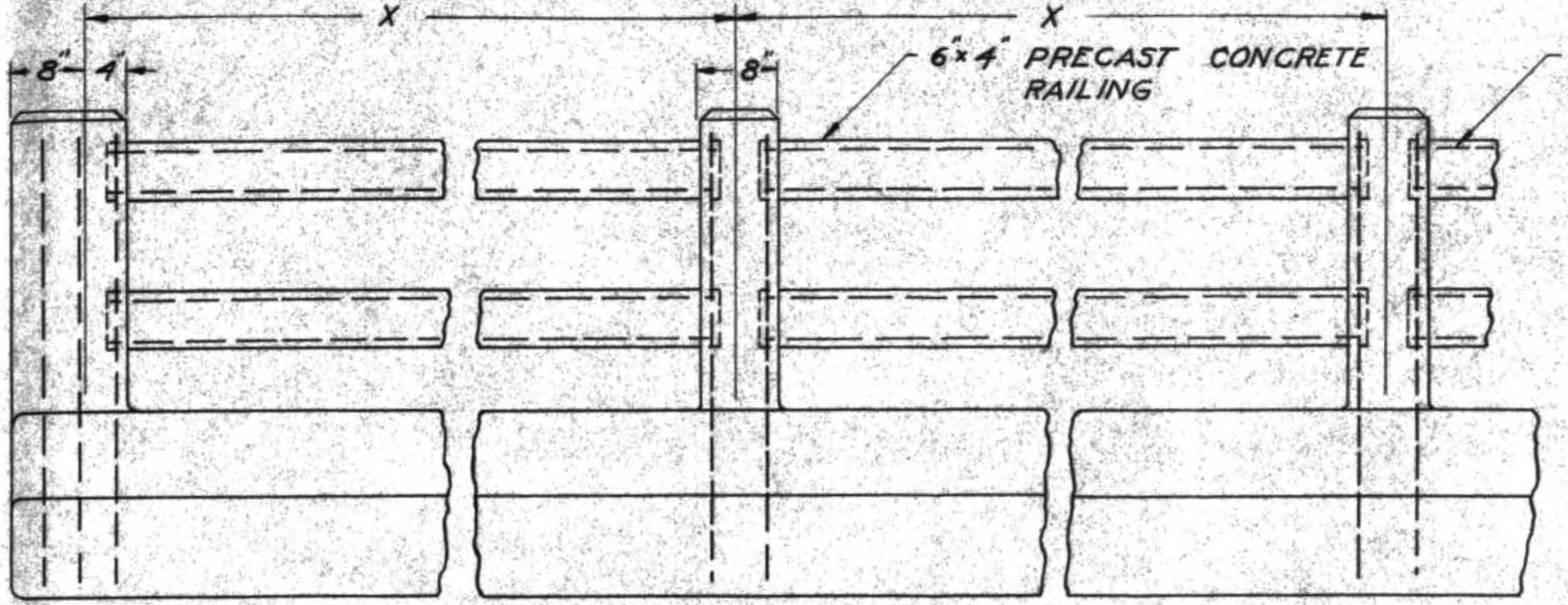
As in case of 30' span L.L. + I.L. caried by the external beam $= \left(\frac{3.125 - 2.08}{3.125}\right)$(L.L. + I.L.)

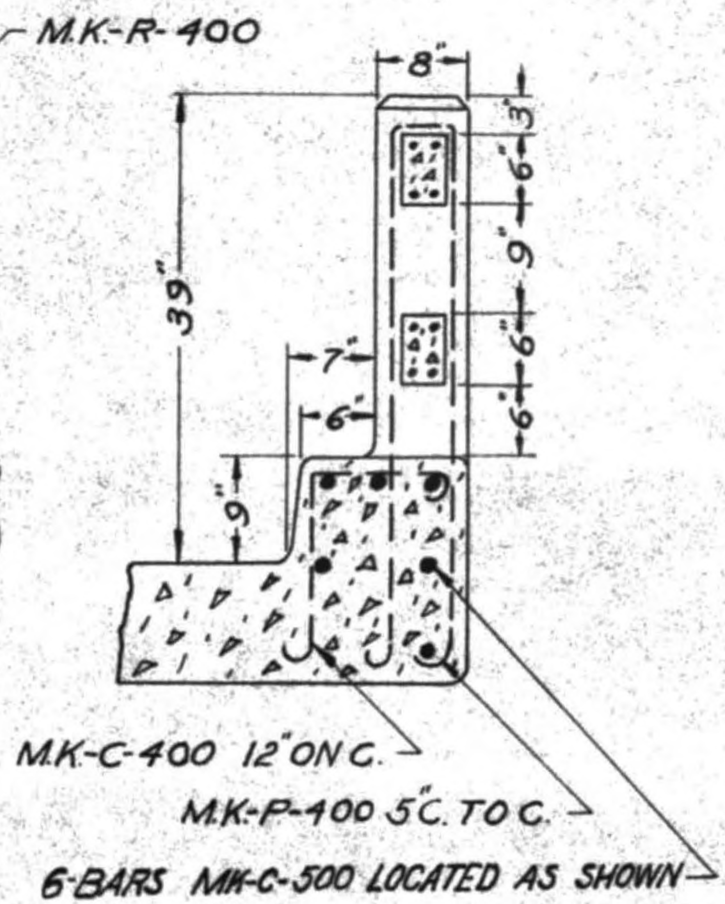
$= .34(9300 + 9300 \times .3)18.6 \times .694 = 57400^{\#'}$ or $688000^{\#''}$

Total M on external beam $= 1,199,000 + 688,000 = 1,887,000^{\#''}$

which is less than the moment on the internal beam.

Same size beam was used as the interior beam.





DETAIL OF RAILING AND CURB

VALUE OF X FOR THE 30' SPAN= 9'-4"

VALUE OF X FOR THE 40' SPAN= 9'-6"

VIEW SHOWING REINFORCING STEEL IN THE FLOOR SLAB

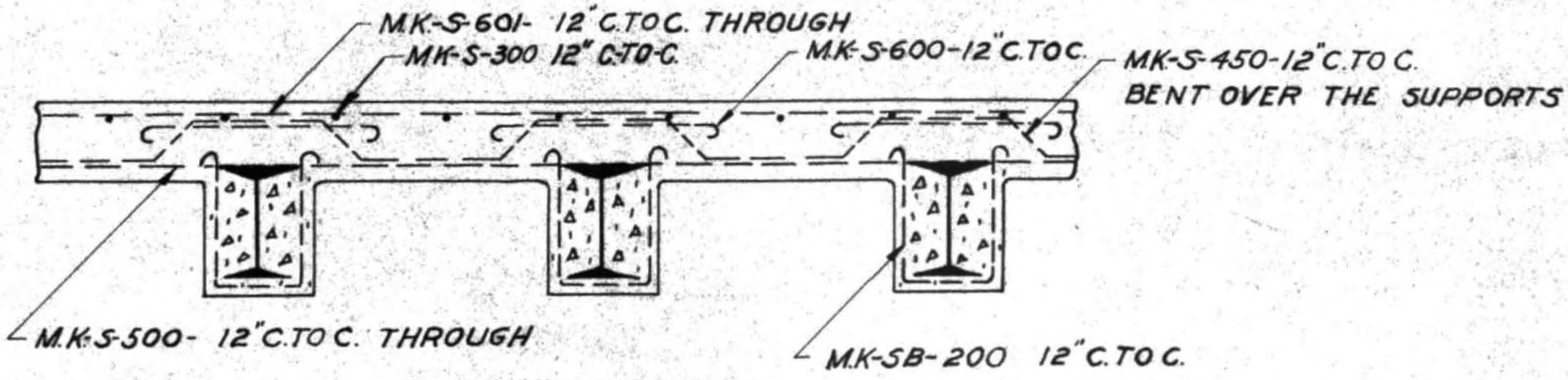
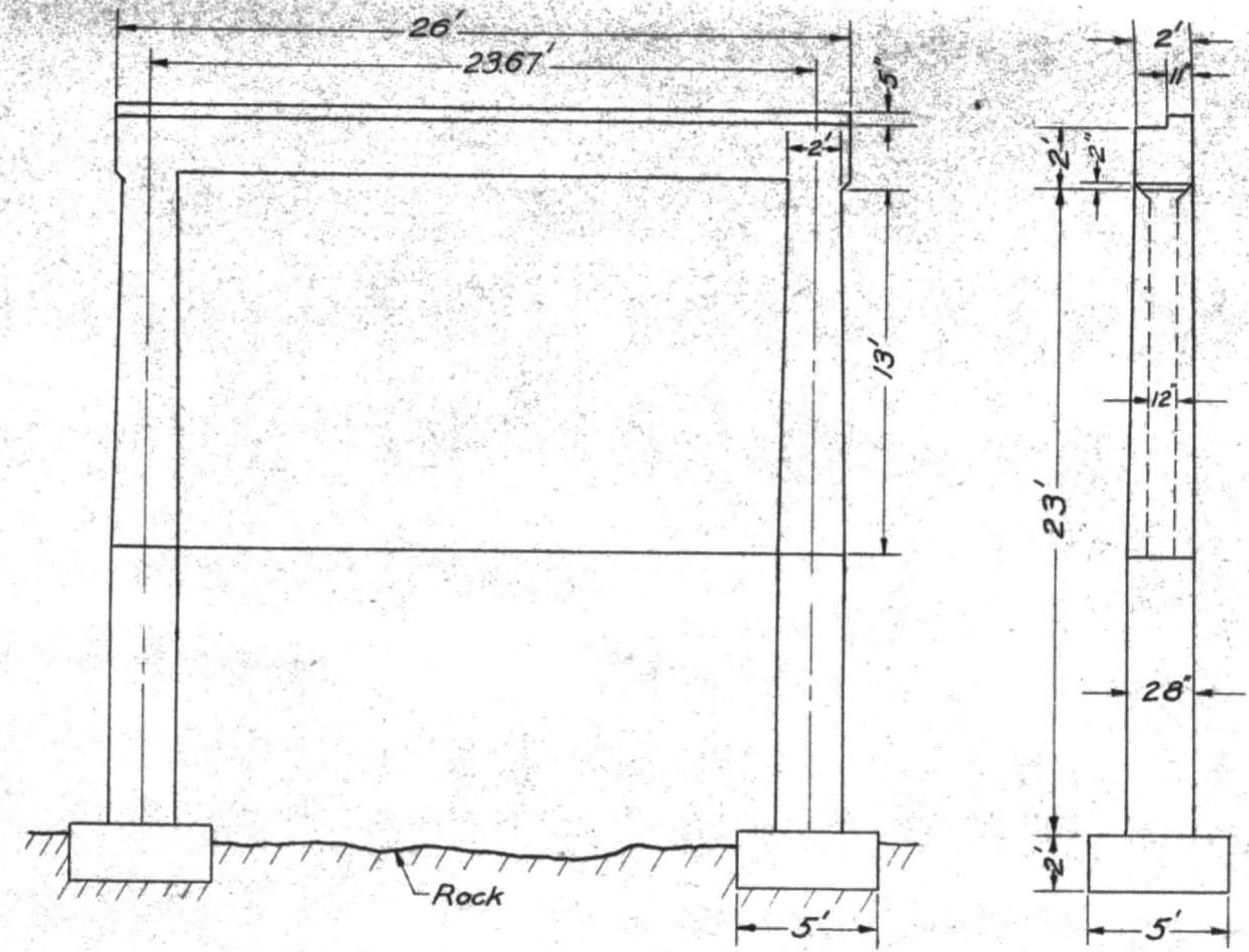


FIGURE-8



PIER 1 & 2 FOR ENCASED I BEAM BRIDGE
FIGURE 9 SCALE 3/16" = 1'-0"

Design of Pier No. 1 and 2

Internal beam loads (using the reactions due to LL which produces maximum moment in the beam) the error is small and on the side of safety.

LL from 30 foot span beam = 6800# (Page 27)

IL from 30 foot span beam = 2040#

DL from 30 foot span beam = 9045# (Page 31)

Total load from 30' span beam $= \frac{6800 + 2040 + 9045}{1000} = 17.9$ Kp

External beam 30 foot span

LL $= .34 \times 6800$ = 2310# (Page 31)

IL $= 2310 \times .3$ = 690#

DL = 7850# (Page 32)

Total load from 30 foot external beam $= \frac{2310 + 690 + 7850}{1000} = 10.8$ Kp

Internal 40 foot span beam

LL = 9300# (Page 33)

IL = 2790#

DL $685 \times \frac{40}{2}$ = 13700# (Page 33)

Total load from 40 foot internal beam $= \frac{9300 + 2790 + 13700}{1000} = 25.9$ Kp

External 40 foot span beam

LL $= (.34)(9300)$ = 3160#

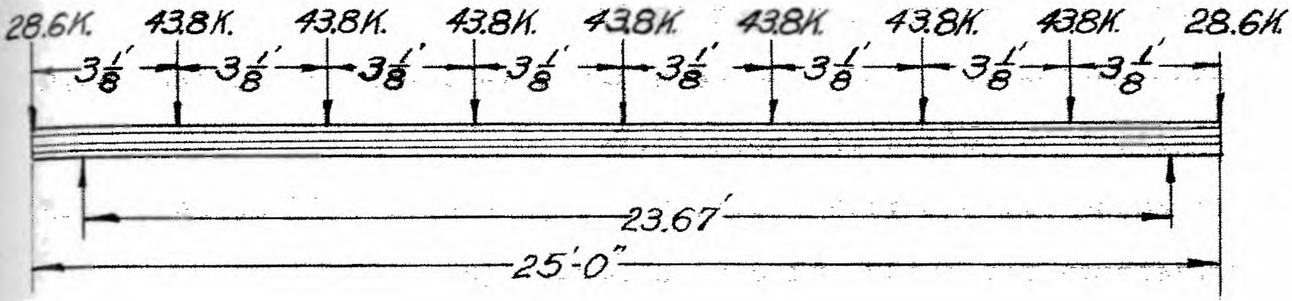
IL $= .3 \times 3160$ = 950#

DL $= 685 \times \frac{40}{2}$ = 13700#

Total load from 40 foot external beam $= \frac{3160 + 950 + 13700}{1000} = 17.8$ Kp

Total load acting on pier from internal beam $= 17.9 + 25.9 = 43.8$ Kp

Total load acting on pier from external beam $= 10.8 + 17.8 = 28.6$



Vertical Forces Acting on the Transverse Beam of Pier.

Total load from the 30' span = 18.0 x 7 + 10.8 x 2 = 147.6 Kp

Total load from the 40' span = 25.8 x 7 + 17.81 x 2 = 207.0 Kp

Assuming pedestal pier as shown in fig. 9

Weight of transverse beam = 2 x 2.225 x 26 x 150 = 17.5 Kp

Weight of diaphragm = 21.6 x 1 x 13 x 150 = 42.0 Kp

Weight of column from top to the bottom of diaphragm

$(2.17)^2$ x 13 x 150 = 9.2 Kp

Weight of column from the bottom of diaphragm to the top of footing = $(2.33)^2$ x 10 x 150 = 8.15 Kp

Weight of footing = $(5)^2$2 x 150 = 7.5 Kp

Lateral Forces Acting on the Pier

Wind load = (projected area on vertical plane)x1.5 x 30

Wind load on pier = 2.17 x 16 x 1.5 x 30 = 1.56 Kp acting about 16' from the bottom of footing.

Wind load on 30' span = 2 x 30 x 1.5 x 30 = 2.72 Kp acting about 29' from the bottom of footing

Wind load on railing = 35 x 1.5 x 30 = 1.58 Kp acting about 29' from the bottom of footing.

Design of Pier 1 and 2 cont'd.

Wind load on 40' span = 3 x 40 x 1.5 x 30 = 5.4 Kp acting about 27.5' from the bottom of footing .

Wind load on Railing = 47 x 1.5 x 30 = 2.1 Kp acting about 27.5' from the bottom of footing.

Wind load on live load = 200# per linear foot and its point of application is 6 ft. above the roadway.

Wind load on 30' span = 200 x 30 =6 Kp acting about 35' from the bottom of footing.

Wind load on 40' span = 200 x 40 =8 Kp acting about 35' from the bottom of footing.

Maximum velocity of the stream at high water is about 9' per second. $F = 1.24AV^2$ (Waddell Vol. I)

F = Total Force in pounds.

A = Area in square ft. perpendicular to the line of motion of water.

V = Velocity of water in feet per second.

$F = 1.92 \times 10 \times (9)^2 = 1.5$ Kp acting about 16' from the bottom of footing.

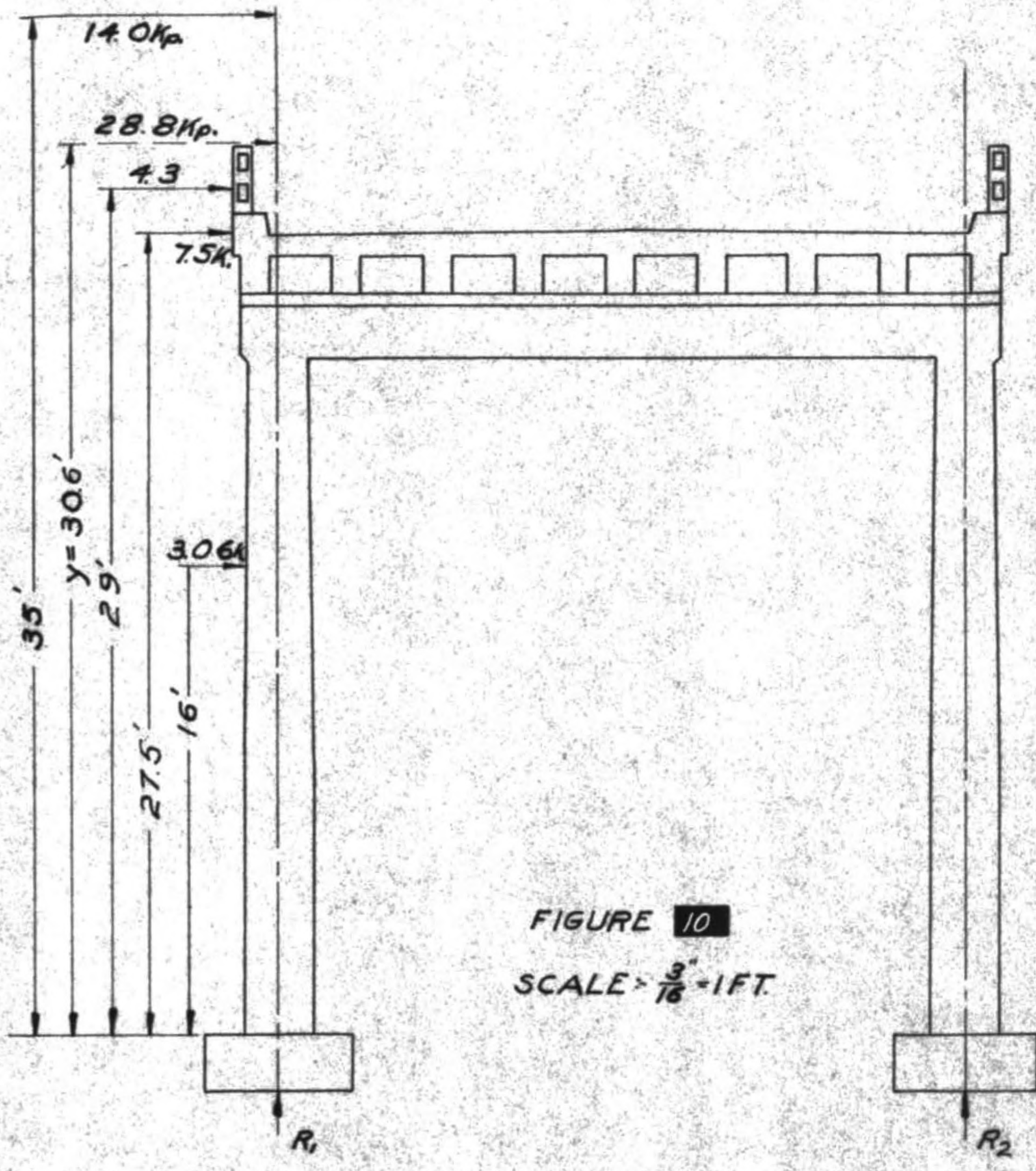


FIGURE 10

SCALE = $\frac{3''}{16}$ = 1 FT.

$\Sigma M_{R_1} = R_2 \times 23.67 - 14 \times 35 - 4.3 \times 29 - 7.5 \times 27.5 - 3.06 \times 16$

$R_2 = 37.1$

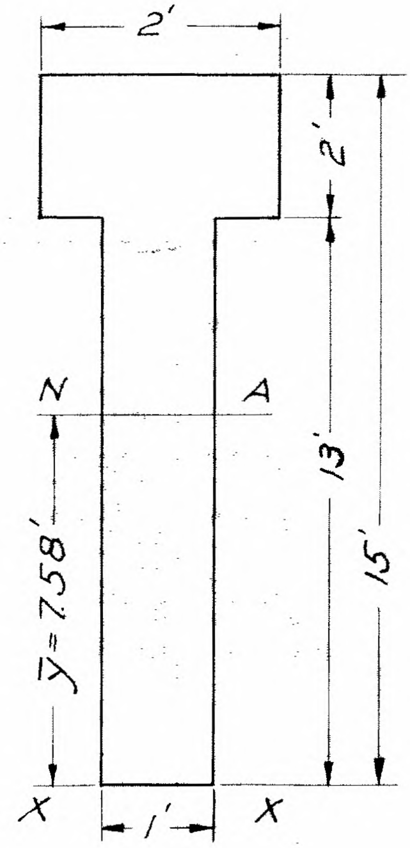
MAGNITUDE OF RESULTANT FORCE =

$\Sigma F = 3.06 + 7.5 + 4.3 + 14 = 28.8$

LATERAL FORCES ACTING ON THE PIER

To locate the neutral axis of the section through the beam and diaphragm, take moment of area about x - x.

$$\bar{Y} = \frac{2 \times 2 \times 14 + 13 \times 1 \times 6.5}{18} = 7.58 \text{ ft.}$$



To find the moment of inertia of the section about N.A.

$$I_B = \frac{b_1 h_1^3}{12} + A_1 d_1^2 + \frac{b_2 h_2^3}{12} + A_2 d_2^2$$

$$I_B = \frac{2^4}{12} + 4 \times (6.42)^2 + \frac{1 \times (13)^3}{12} +$$

$$13 \times (1.08)^2 = 364.5$$

$$I_B = 364.5 \text{ ft}^4$$

Moment of inertia of column section at the base of diaphragm

$$I_c = \frac{(2.33)^4}{12} = 2.46 \text{ ft}^4.$$

$$\text{Stiffness factor for column} = \frac{I_c}{L} = \frac{2.46}{10} = .246 = F_c$$

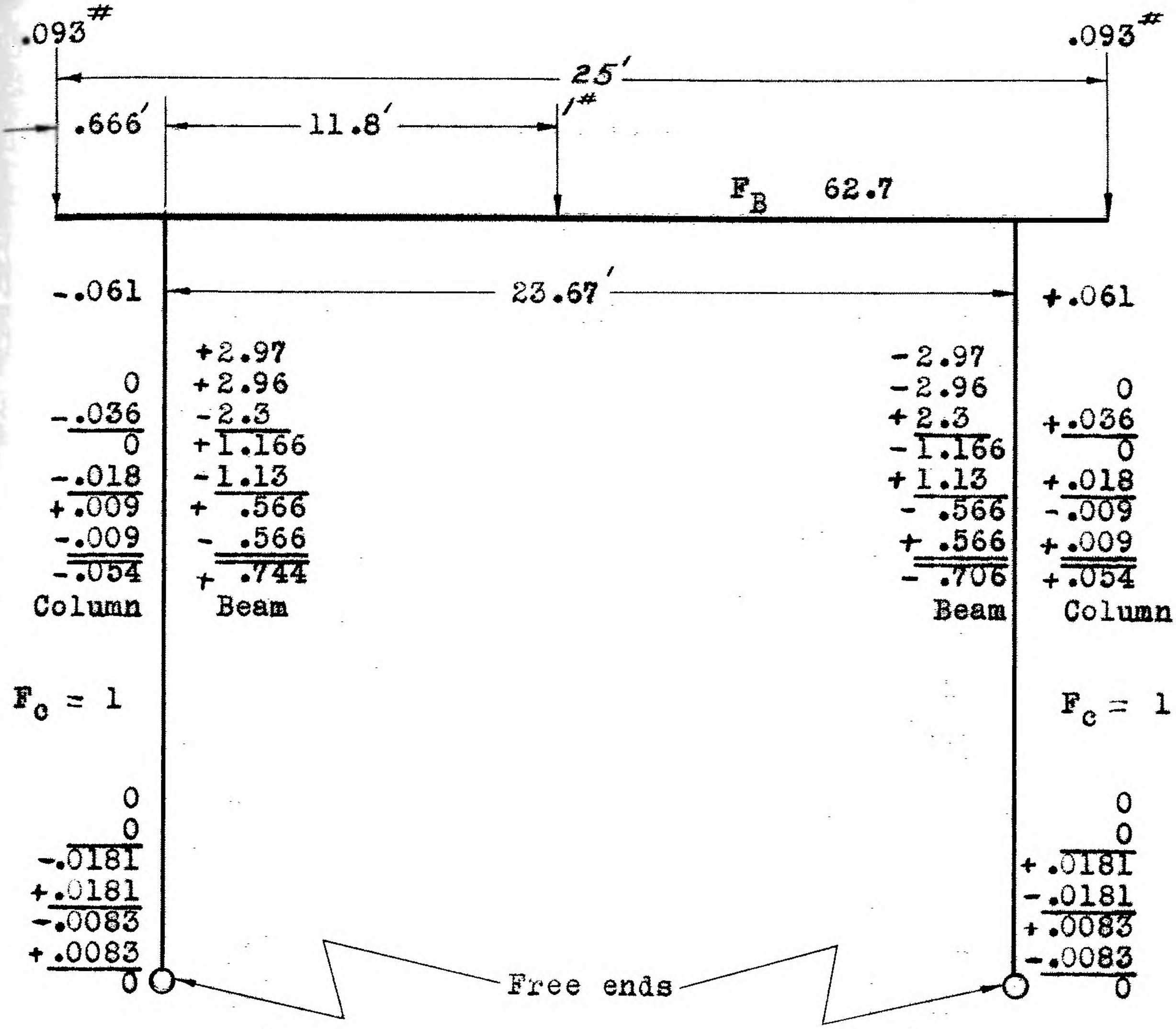
$$\text{Stiffness factor for beam} = \frac{I_B}{L} = \frac{364.5}{23.67} = 15.4 = F_B$$

Dividing both stiffness factors by .246,

$$F_B = \frac{15.4}{.246} = 62.7 \qquad F_c = \frac{.246}{.246} = 1$$

Design of Pier 1 and 2

Moment Distribution for Determining Moment in Column due to CL



$$M_{FAB} = M_{FBA} = \frac{PL}{2}\left(\frac{\frac{L}{2}}{L}\right)^2 = \frac{PL}{8} = \frac{P\,23.67}{8} = 2.97P$$

$P = 7 \times 43.8 = 306.6$

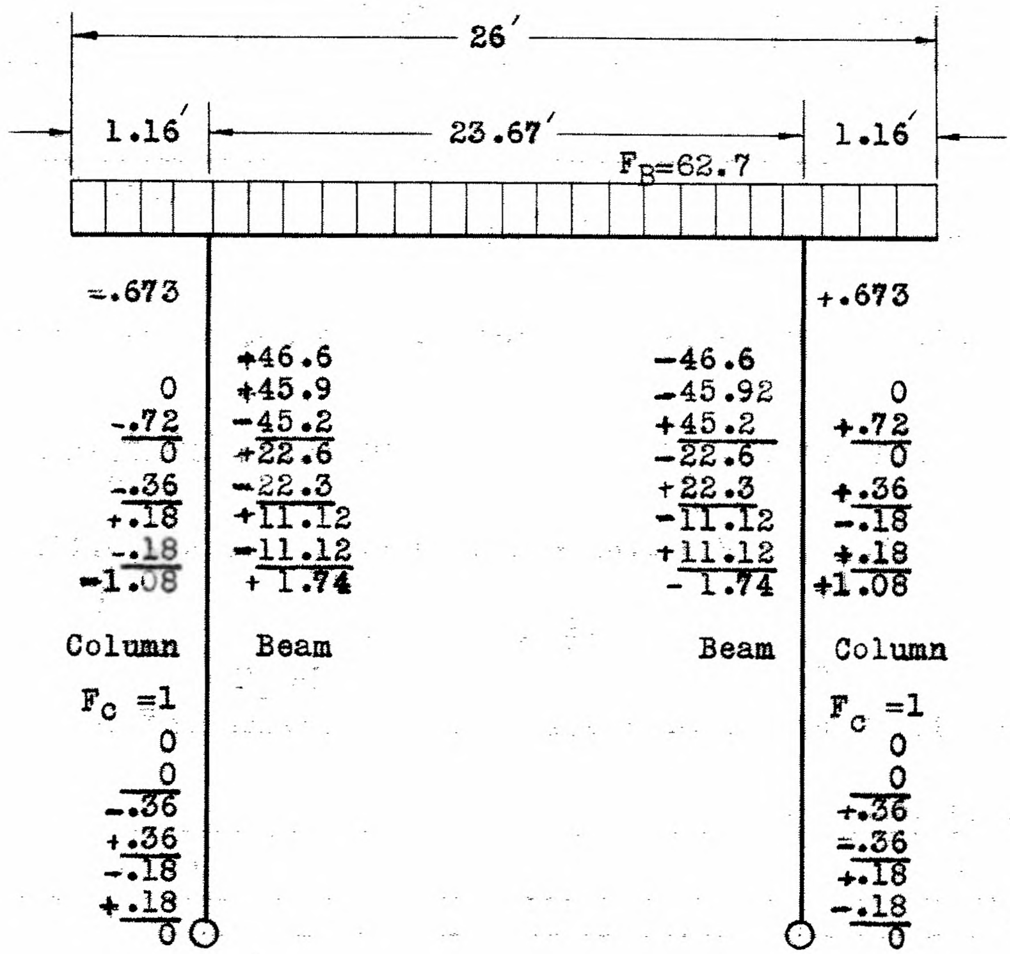
Moment in the column at beam $= .054P = .054 \times 306.6$

Moment in the column at beam = 16.6 Kp Ft.

Design of Pier 1 and 2

Moment distribution for determining the moments due to uniform load.

Let w = unit load per foot of length.



$$M_{FAB} = M_{FBA} = \frac{wL^2}{12} = \frac{w(23.67)^2}{12} = 46.6w$$

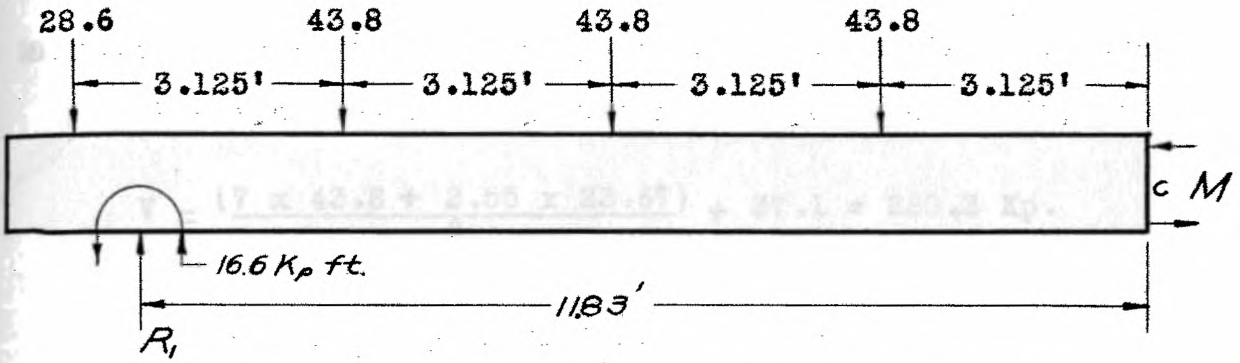
$$w = \frac{(2 \times 2 + 13 \times 1)(150)}{1000} = 2.55 \text{ Kp per foot of length.}$$

Moment in the column at the beam = 1.08w

Moment in the column at the beam = 2.73 Kp ft.

Design of Pier 1 and 2

Determining the moment in transverse beam due to loads from beams



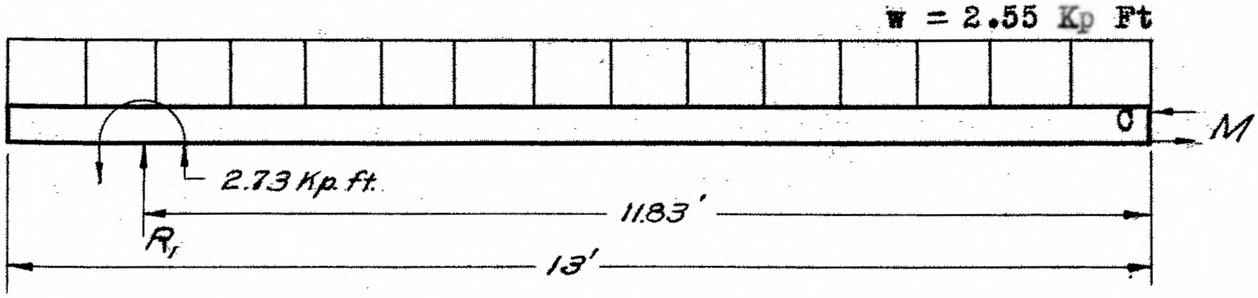
Reaction from column on the beam $\quad R_1 = \frac{1}{2}\Sigma F$ on beam

$R_1 = \frac{147.6 + 207}{2}$ (Page 39) $\quad R_1 = 177.3$ Kp.

$$\Sigma M_c = 131.4 \times 6.25 + 28.6 \times 12.5 + 16.6 + M - 177.3 \times 11.83 = 0$$

$$M = 919 \text{ Kp Ft.}$$

Determining the moment in transverse beam due to DL of beam



$$R_1 = \frac{26 \times 2.55}{2} = 33.1 \text{ Kp}$$

$$\Sigma M_c = M + \frac{WL^2}{8} + 2.73 - 33.1 \times 11.83$$

$$M = 33.1 \times 11.83 - 2.73 - \frac{2.55 \times (13)^2}{8}$$

$$M \text{ due to DL} = 334.2 \text{ Kp Ft.}$$

Design of Pier 1 and 2

Total moment at the center = 919.0 + 334.2 = 1253.2 Kp Ft.

$$A_s = \frac{M}{f_s J d} = \frac{1252.3 \times 1000 \times 12}{18000 \times .87 \times 177} = 5.45 \text{ in}^2.$$

Maximum shear is over the support on the down stream column when the wind load and water pressure are considered.

$$V = \frac{(7 \times 43.8 + 2.55 \times 23.67)}{2} + 37.1 = 220.3 \text{ Kp.}$$

$$\leq 0 = \frac{V}{uJd} = \frac{220.3 \times 1000}{120 \times .87 \times 177} = 12 \text{ in:}$$

Use 3 – $1\frac{1}{4}$ in. square bars MK – D1050 4 in. C to C.

A_s used = 4.69 ≤ 0 = 15 in.

Use 1 – $1\frac{1}{8}$ in. square bar MK – D –950

A_s used = 1.27 ≤ 0 use = 4.5 in.

Total A_s used = 5.96 ≤ 0 use = 19.5 in.

Check for Shear

$$v = \frac{V}{b^1 J d} = \frac{220.3 \times 1000}{12 \times .87 \times 177} = 121$$

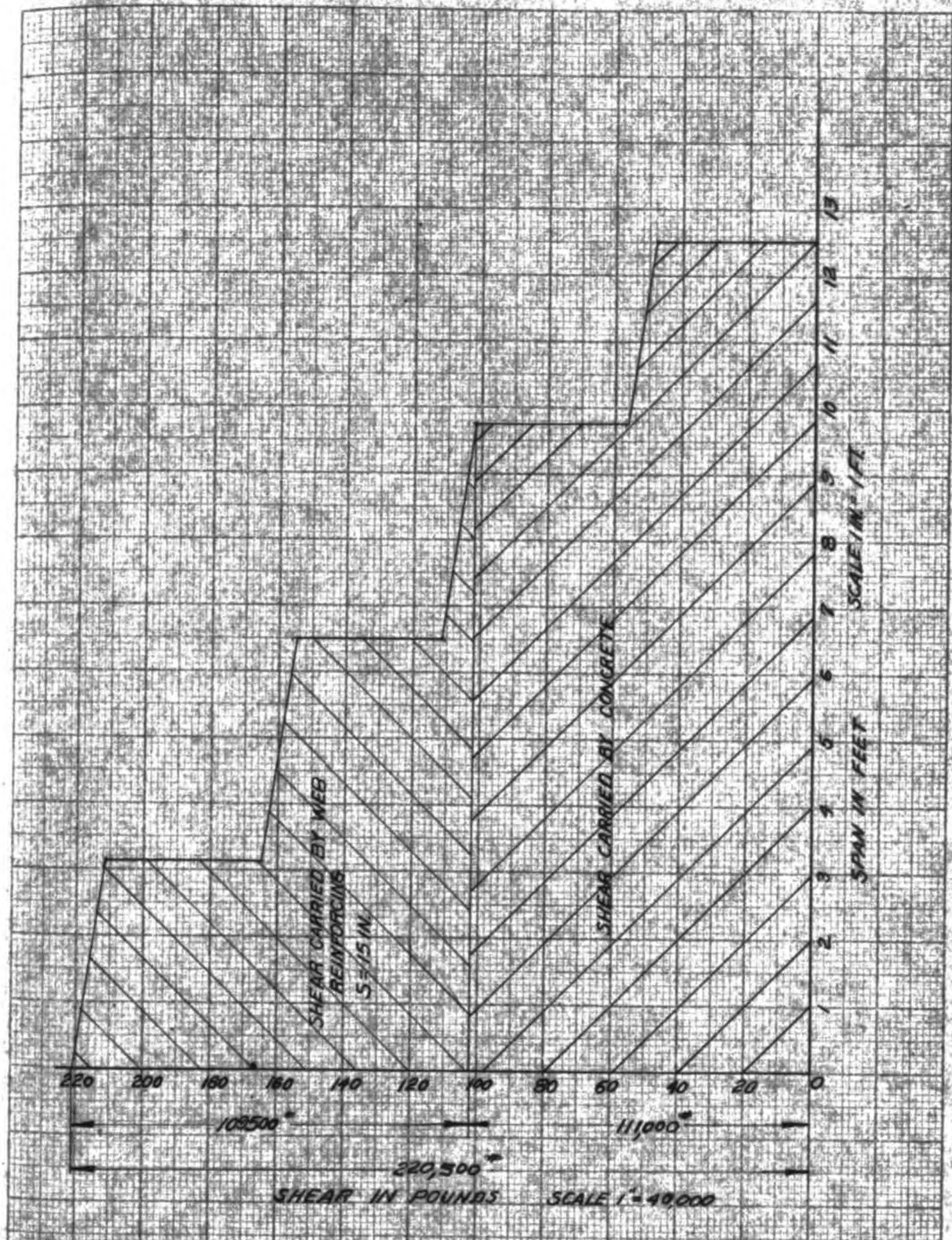
Since value of v is between $.03f_c^1$ and $.06f_c^1$ web reinforcing is used. $V_c = v b^1 J d$ = 60 x 12 x .87 x 177 = 111,000#.

$$V_s = V - V_c' = 220.3 \times 1000 - 110800 = 109{,}300\#.$$

Let S = spacing of web reinforcing bars. Using $\frac{5}{8}$ round bars,

$$A_s = .62 \text{ in}^2 \quad S = \frac{A_s f_s J d}{V_s} = \frac{.62 \times 18000 \times .87 \times 177}{109000} = 15.8$$

Use bars MK – PB – 500 15 in. C to C full length of beam.


SHEAR CARRIED BY WEB
REINFORCING
S=15 IN.
SHEAR CARRIED BY CONCRETE
0
1
2
3
4
5
6
7
8
9
10
11
12
13
SPAN IN FEET
SCALE 1 IN = 1 FT.
220
200
180
160
140
120
100
80
60
40
20
0
109500#
111,000#
220,500#
SHEAR IN POUNDS
SCALE 1" = 40,000

Design of Pier 1 and 2

Check for Double Reinforcing

$$M = 1253.2 \times 1000 \times 12 = 15{,}050{,}000 \text{ in pounds}$$

$$M_1 = Kbd^2 = 208.3 \times 12 \times 177 = 78{,}200{,}000 \text{ in.pounds.}$$

Since M_1 is greater than M compression steel is not required. However, the mimimum steel required was used.

$$A_s = .005 \times 24 \times 24 = 2.88 \text{ in}^2.$$

Use 2 - bars MK - PB - 700 - 5 in. C to C. (See fig.12 page 55)

Use 2 - bars MK - PB - 701 - 6 in. C to C.

Use 2 - bars MK - PB - 702 - 18 in. C to C.

$$\text{Total } A_s \text{ used} = 3.61 \text{ in}^2.$$

Steel in diaphragm must not be less than .125 in^2 per foot of height.

Use 21 - bars MK - DV - 400 12 in. C to C alternating front and back.

Use 13 - bars MK - DH - 400 12 in. C to C alternating front and back. (See fig.12 page 55)

Column Design

Working Stresses Used in Columns

J. C. S. Rules

$$f_c = .3f'_c = 900 \ \#/in^2.$$

p must be between .005 and .04 of cross section area.

$$f_s = 16000 \ \#/in^2.$$

$$n = 10$$

The dangerous section in the column due to flexural stresses is at the base of the diaphragm. Moment on that section is the sum of moments due to following loads:

(a) Wind loads + (b) Water pressure on pier + (c) Dead load of girder + (d) Live and dead load from spans.

Total wind and water pressure = 28.8 Kips.

Moment on column $M_{a+b} = \frac{28.8}{2} \times 12.8 = 185.4$ K. Ft.

Moment on column (c) = 2.73 K. Ft.

Moment on column (d) = 16.6 K. Ft.

Total M = 204.7 K. Ft.

Total vertical force = $\frac{\text{Load from spans}}{2} + \frac{\text{Dead load of Pier}}{2} +$ Reaction due to wind load.

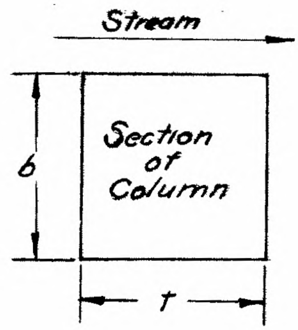
$$\text{Total vertical force } N = \frac{147.6 + 207}{2} + \frac{17.5 + 42 + 9.2}{2} + 37.1$$

$$N = 211.7 \text{ Kp.}$$

Column Design

Ratio of unsupported length L to the smallest sectional dimension must be less than 15.

$$\frac{L}{b} = \frac{120}{28} = 4.3$$



$$X_0 = \frac{M}{N} = \frac{204.7 \times 12}{211.7} = 11.6$$

$$b = t = 28 \text{ in.}$$

$$\frac{t}{X_0} = \frac{28}{11.6} = 2.41$$

$$\frac{NX_0}{f_c b t^2} = \frac{211.7 \times 11.6}{900 \times (28)^2} = .125$$

From diag. 5 np = .13 Therefore p = .013

A_s = .013 x area of section = .015 x $(28)^2$

A_s = 10.2 in^2. Use 2 - $1\frac{1}{8}$ square bars. MK - C - 950. 7 in. C. to C. at the footing and 6 in. C to C at the top, in each b-face.

Use 4 - $1\frac{1}{4}$ square bars. MK - C - 1050 bars in corners. 21 in. C to C at the footing, and 18 in. C to C at the top, the bars hooked at both ends.

Lateral ties or hoops are $\frac{1}{4}\phi$ bars MK - C - 200 and spaced 12 in. C.C. for the full height of the column.

Longitudinal Forces on the Pier

According to the Georgia State Highway specifications, a longitudinal force is equal in magnitude to .1 of the live load on the bridge. This force is applied 4 feet above the floor.

LL on the 30' span = one 15 ton truck = 30 Kp.

LL on the 40' span = one 15 ton truck and the front wheels of a $11\frac{1}{4}$ ton truck.

LL on the 40' span = 30 Kp + 4.5 Kp = 34.5 Kp.

Total load from both spans on pier = (34.5 + 30).1 = 6.45 Kp.

Maximum moment on the pier due to this load will occur above footing, its magnitude $M = \frac{6.45 \times (25' + 1.15'' + .75 + 4')}{2}$

$$M = 3.22 \times 31.5' = 101 \text{ Kp Ft.}$$

This moment is less than moment in the faces at right angles to the center line of the stream. However, same size and number of bars were used as in face - b . .

Design of Footing

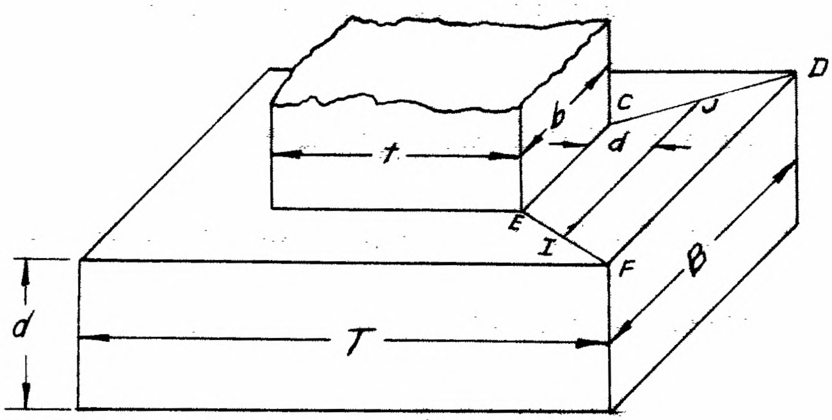


FIGURE 11

Allowable Working Stresses

$f_c = .4f'_c$ = 1200 #/in^2.

$n = 10$

$f_s = 18000$ #/in^2.

$K = 208.3$

$v = .02f'_c$ No special anchorage required = 60 #/in^2.

$v = .03f'_c$ Special anchorage required = 90 #/in^2.

$u = .375f'_c$ Bars two ways required = 112 #/in^2.

$v_p = .06f'_c$ punching shear = 180 #/in^2.

Let W = weight of footing

N = total load on footing

L = bearing power of soil in tons per ft.^{2}.

Assuming B = T = 5 ft., and d = 21" d' = 3",

$$W = 5 \times 5 \times 2 \times 150 = 7500 \#.$$

Design of Footing

$$Y = \text{upward thrust} = \frac{N}{BT}$$

Total N = 211.7 + weight of column above footing = 211.7 + 8.15
= 219.8 Kp.

$$Y = \frac{219.8 \times 1000}{5 \times 5} = 8800 \ \#/ft^2.$$

M = moment along CE (fig. 10) = $(T - t)^2(2B + b)\frac{Y}{24}$

$$M = (5 - 2.33)^2(2 \times 5 + 2.33)\frac{8800}{24}$$

$$M = 32300'\#$$

P = punching shear along CE (Page 52) = $(B + b)(T - t)\frac{Y}{4}$

$$P = (5 + 2.33)(5 - 2.33)\frac{8800}{4}$$

$$P = 43000$$

$$d = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{32300 \times 12}{208.3 \times 28}}$$

$$d = 8.15 \text{ in.}$$

$$d = \frac{P}{12 v_p t} = \frac{43000}{12 \times 180 \times 28}$$

$$d = .72 \text{ in.}$$

Check for Design Without Steel

Let f_t = unit tensile strength of the concrete. f_c must not exceed $.01 f_c$ or f_t must not exceed 30 $\#/in^2$.

$$f_t = \frac{6M}{td^2} = \frac{6 \times 32300 \times 12}{28 \times (21)^2}$$

$f_t = 189 \ \#/in^2$. Therefore steel must be used.

Design of Footing

$$A_s = \frac{M}{f_s jd} = \frac{32300 \times 12}{18000 \times .87 \times 21}$$

$$A_s = 1.18 \text{ in}^2.$$

Steel in B or T direction is distributed over a width of B or T unless B or T is greater than (t + 2d) or (b + 2d)

$B = T = 5'$ $(t = b)$ $t + 2d = 2.33 + \frac{2 \times 21}{12} = 5.83'$

Therefore steel is distributed over a distance B or T.

Check for Shear

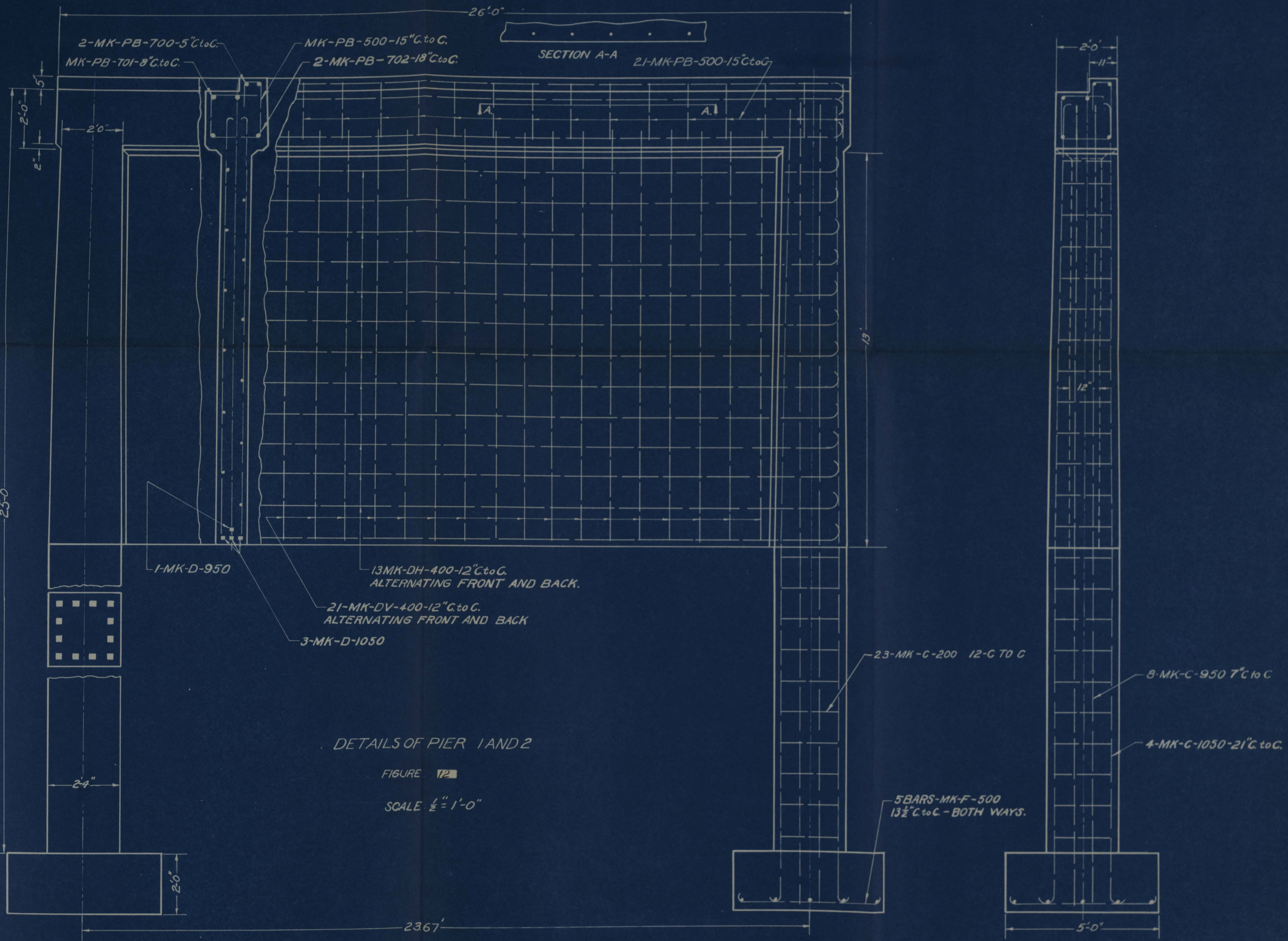
$$v = \frac{V_1}{12IJjd}$$

Where V_1 = Area of (IJDF)Y (see fig. on page 52)

Since $\frac{B-b}{2} > d$ or $\frac{T-t}{2} > d$, $v = 0$

Therefore $v = 0$. Therefore no special anchorage is required.

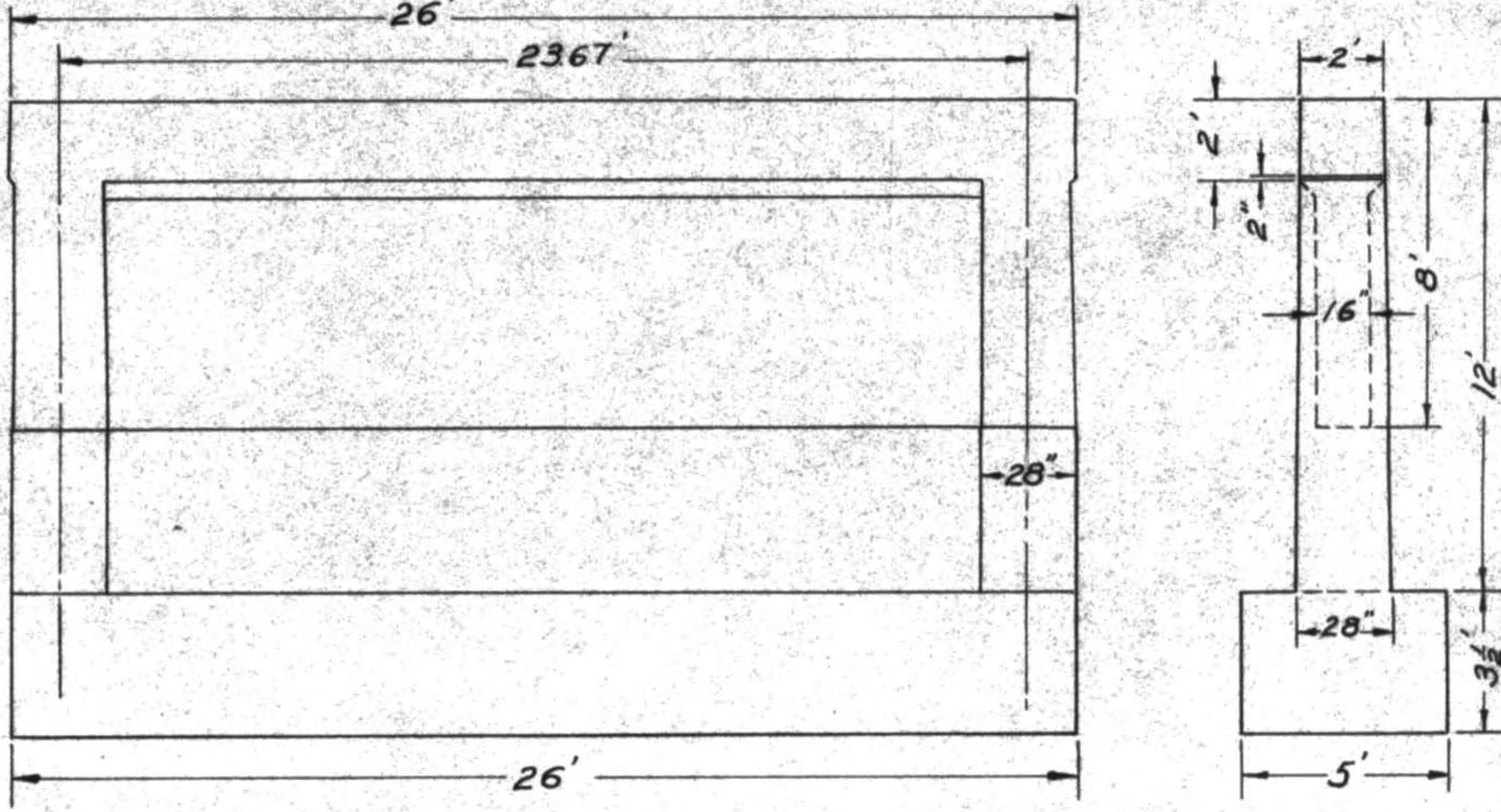
Use 5 bars MK - F - 500. $13\frac{1}{2}$ in. C to C in both directions.



DETAILS OF PIER 1 AND 2

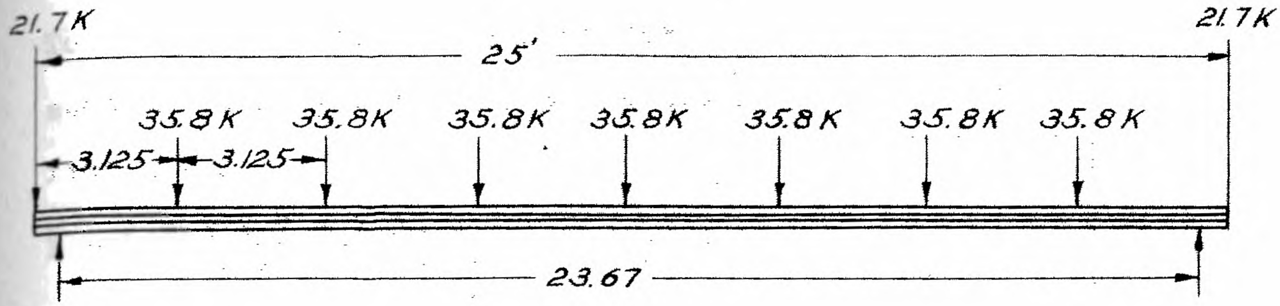
FIGURE 12

SCALE ½" = 1'-0"



PIER NO 3
FIGURE 13
SCALE $\frac{1}{4}$" = 1'-0"

Design of Pier No. 3



Using the reaction due to LL which gives maximum moment in the beam the error is small and on the side of safety.

Load per internal beam per span = 17.88 Kp.

Load per internal beam per two spans = 35.76. Use 35.8 Kp.

Load per external beam per span = 10.85 Kp.

Load per external beam per two spans = 21.7 Kp.

Assuming pier as shown in fig. 13 page 56

Weight of transverse beam $= \frac{2 \times 2.25 \times 26 \times 150}{1000} = 17.6$ Kp.

Weight of diaphragm $= \frac{21.6 \times 1.33 \times 6 \times 150}{1000} = 26$ Kp.

Weight of columns $= \frac{2.17 \times 2.17 \times 12 \times 150 \times 2}{1000} = 17.0$ Kp.

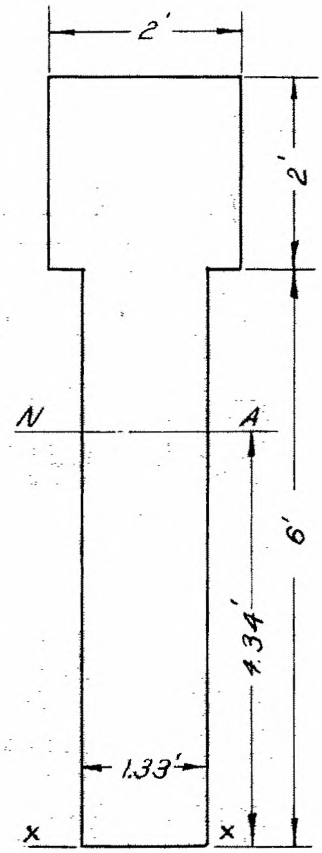
Total DL = 17.6 + 26 + 17 = 60.6 Kp.

Assume footing 3.5 x 5 ft. wide,

Weight of footing $= \frac{3.5 \times 5.0 \times 26 \times 150}{1000} = 68.5$ Kp.

Design of Pier No. 3

To locate the neutral axis of the section, taking moment of areas about line x - x,



$$\bar{Y} = \frac{2 \times 2 \times 7 + 1.33 \times 6 \times 3}{2 \times 2 + 1.33 \times 6} = 4.34 \text{ ft.}$$

$$I_B = \frac{2 \times 2^3}{12} + 4 \times (2.66)^2 + \frac{1.33 \times 6^3}{12}$$

$$+ 8 \times (1.34)^2$$

$$I_B = 67.8 \text{ ft}^4.$$

$$I_C = \frac{(2.25)^4}{12} = 2.14 \text{ ft}^4.$$

$$\text{Stiffness factor for beam} = \frac{I_B}{23.67} = \frac{67.8}{23.67}$$

Stiffness factor for beam = 2.8

$$\text{Stiffness factor for column} = \frac{I_c}{L} = \frac{2.14}{4}$$

$$= .535 \text{ Ft}^3.$$

Assume footing 5 ft. wide and 3.5 ft. deep.

$$\text{Therefore, } I_f = \frac{5 \times (3.5)^3}{12} = 17.9 \text{ ft}^4.$$

$$\text{Stiffness factor for footing} = \frac{17.9}{23.67} = .76 \text{ ft}^3.$$

Dividing stiffness factors by .535,

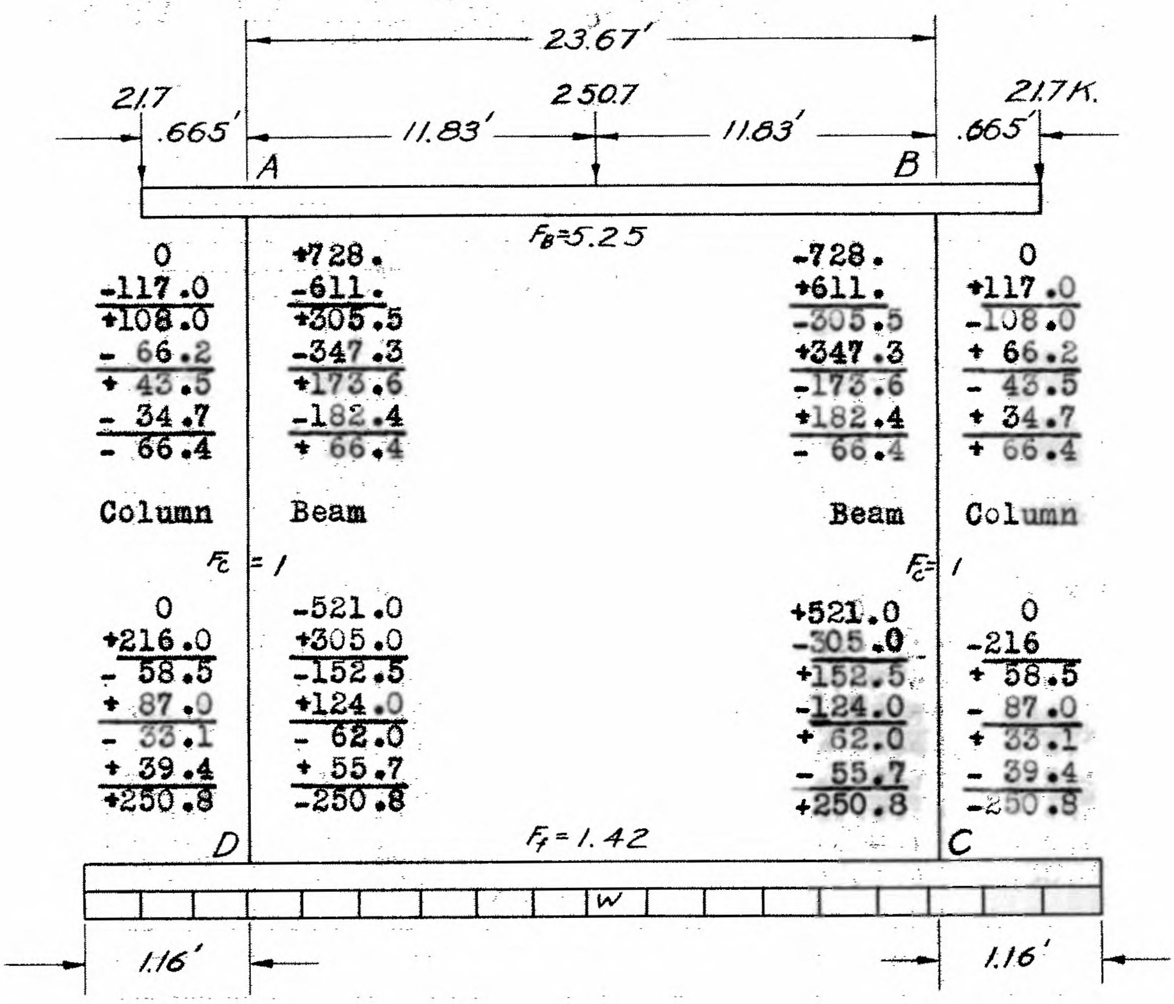
$$F_B = \frac{2.8}{.535} = 5.25 \text{ ft}^3$$

$$F_c = \frac{.535}{.535} = 1.0 \text{ ft}^3$$

$$F_f = \frac{.76}{.535} = 1.42 \text{ ft}^3$$

Design of Pier No. 3

Moment Distribution for Concentrated Loading from Beams



Fixed end moments

$$M_{FAB} = M_{FBA} = Pa\left(\frac{b}{L}\right)^2 - 21.7 \times .665$$

$$(a = b = 11.83) = 250.7 \times 11.83\left(\frac{11.83}{23.67}\right)^2 - 21.7 \times .665 = 728$$

$$w = \frac{294.1}{26} = 11.3$$

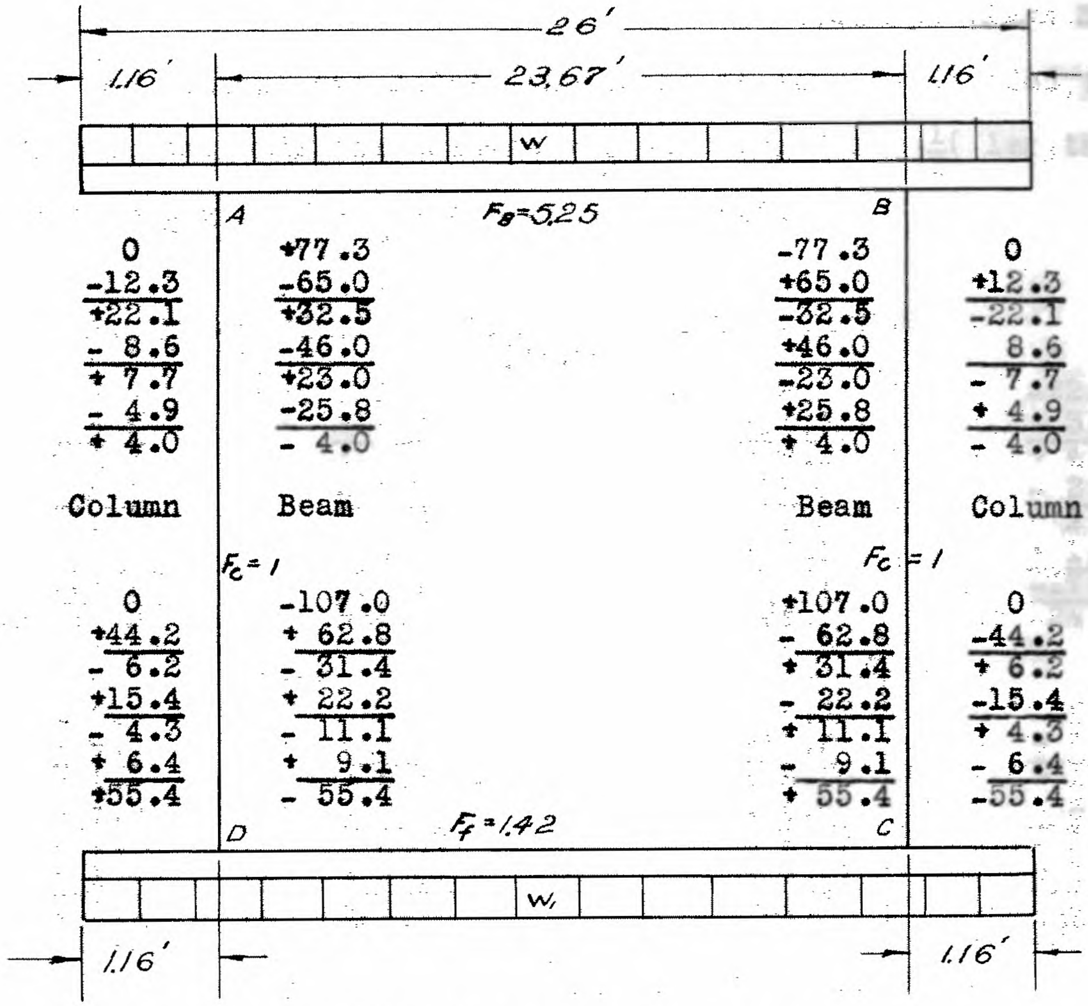
$$M_{FCD} = M_{FDC} = \frac{wL^2}{12} - \frac{w(1.16)^2}{2} = \frac{11.3(23.67)^2}{12} - \frac{11.3(1.16)^2}{2} =$$

521'#

Design of Pier No. 3

Moment Distribution for Uniform Load

$$W = \frac{\text{weight of beam} + \text{weight of diaphragm}}{\text{length of beam}} = \frac{17.6 + 26}{26} = 1.68$$



$$W_1 = W + \frac{\text{weight of column}}{26} = 1.68 + \frac{17}{26} = 2.33$$

$$M_{FAB} = M_{FBA} = \frac{WL^2}{12} - \frac{W(1.16)^2}{2} = \frac{1.68 \times (23.67)^2}{12} - \frac{1.68 \times (1.16)^2}{2}$$

$$= 77.3$$

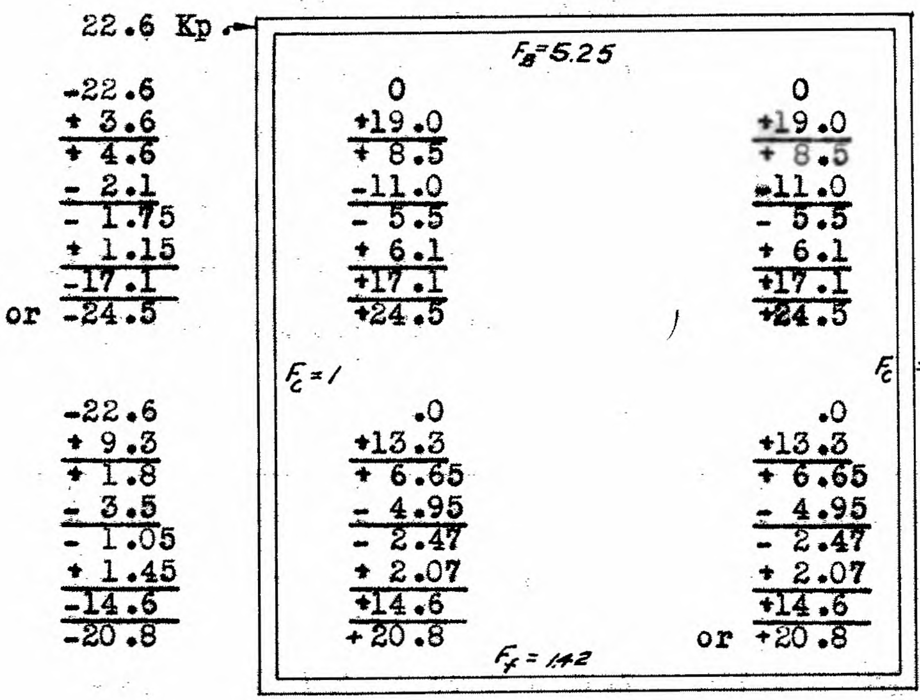
$$M_{FDC} = M_{FCD} = \frac{W_1 L^2}{12} - \frac{W\ (1.16)^2}{2} = \frac{2.3(23.67)^2}{12} - \frac{2.3(1.16)^2}{2} = 107$$

Design of Pier No. 3

Moment Distribution for Wind Load

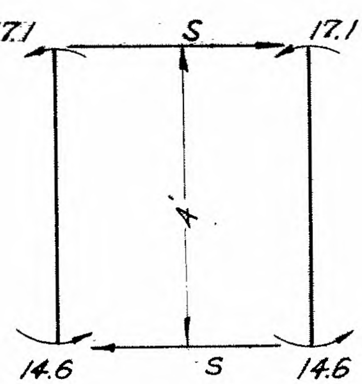
Total transverse force on the pier = 2(6 + 2.72 + 1.58 + 1) =
22.6 Kp. (Page 39)

Since the columns are alike, the shear is equally divided, therefore the moment in the columns = $\frac{1}{2}$(shearing force)$\frac{1}{2}$(length of column) = $\frac{1}{2}$ x 22.6 x $\frac{1}{2}$ x 4 = 22.6 Kp. Feet.



$\Sigma M = 0 = 4S - 2(17.1 + 14.6) = 0$

S = 15.8 Kp.



$M_{AD} = 17.1(\frac{22.6}{15.8}) = 24.5$ Kp. ft. $= M_{AB}$

$M_{DA} = 14.6\ \frac{22.6}{15.8} = 20.8$ Kp. ft. $= M_{DC}$

Design of Pier No. 3

Total M at D = +DLM + CLM + Wind LM.

Total M at D = -55.4 - 250.8 + 20.8.

Total M at D = -285.4 Kp. Ft.

Total M at C = DLM + CLM + Wind LM.

Total M at C = +55.4 + 250.8 + 20.8

Total M at C = +327.0 Kp. Ft.

Total M at A = DLM + CLM + Wind LM.

Total M at A = -4.0 + 66.4 + 24.5

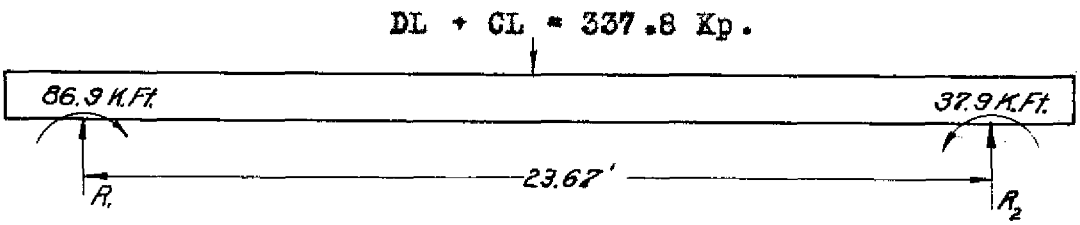
Total M at A in beam = 86.9 Kp. Ft.

Total M at B in beam = DLM + CLM + Wind LM.

Total M at B in beam = +4.0 - 66.4 + 24.5

Total M at B in beam = -37.9 Kp. Ft.

Showing the beam as free body,

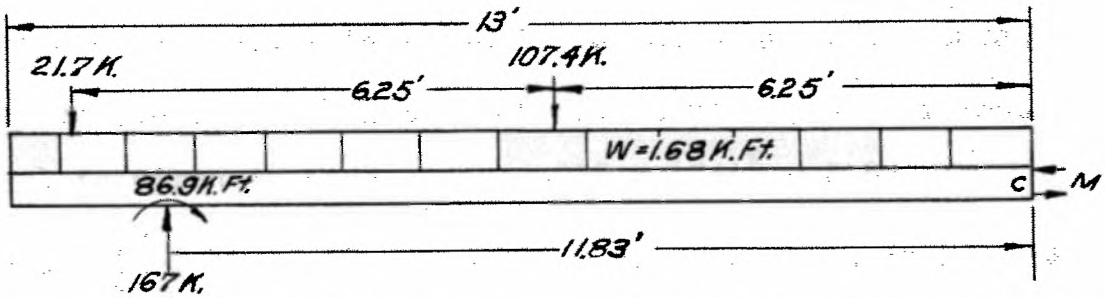


$$\Sigma M_{R_2} = 23.67R_1 + 86.9 - 37.9 - 337.8 \times \frac{23.67}{2} = 0$$

$R_1 = 167.0$ Kp. $\qquad R_2 = 171$ Kp.

Design of Pier No. 3

Positive moment in the beam is greatest at the center of span. Showing the left portion of the beam as free body,



$$\Sigma M_c = M - 86.9 + 107.4 \times 6.25 + 21.7 \times 12.5 - 167 \times 11.83 + 1.68 \times \frac{(13)^2}{2}$$

$$M = 977.0 \text{ Kp. Ft.}$$

$$A_s = \frac{M}{f_s J d} = \frac{977.0 \times 1000 \times 12}{18000 \times .87 \times 92} = 8.18$$

Check for Perimeter

Maximum vertical shear is over the second support R_2 = 171 Kp.

$$\Sigma O = \frac{V}{uJd} = \frac{171 \times 1000}{120 \times .87 \times 92} = 17.85 \text{ in.}$$

Use 4 - $1\frac{1}{4}$" square bars MK - D - 1050 4 in. C to C through, hooked at both ends d = 93'

Use 2 - 1" square bars MK - D - 850 8 in. C to C through, hooked at both ends d = 89'

Total A_s used = 8.25 in^2. ΣO used = 28.0 in.

Design of Pier No. 3

Check for Double Reinforcing

$M = 977.0 \times 1000 \times 12 = 11{,}740{,}000$ in. pounds.

$M_1 = Kbd^2 = 208.3 \times 16 \times (92)^2 = 28{,}200{,}000$ in. pounds.

Compression steel is not required. However, the minimum steel required is used. $A_s = .005 \times (24)^2 = 2.88\ in^2$.

Use 5 - $\frac{7}{8}$" ϕ bars MK - B - 700 placed as shown in fig.14 page 73 and hooked at both ends.

Check for Shear

$$v = \frac{V}{b\,Jd} = \frac{171.0 \times 1000}{16 \times .87 \times 92} = 134\#/in^2.$$

Web reinforcing is required.

$$V_c = vb'Jd = 60 \times 16 \times .87 \times 92 = 76{,}800\#$$

$$V = 171.0 - 21.7 - 1.66 \times 1.68 = 146.5\ Kp.$$

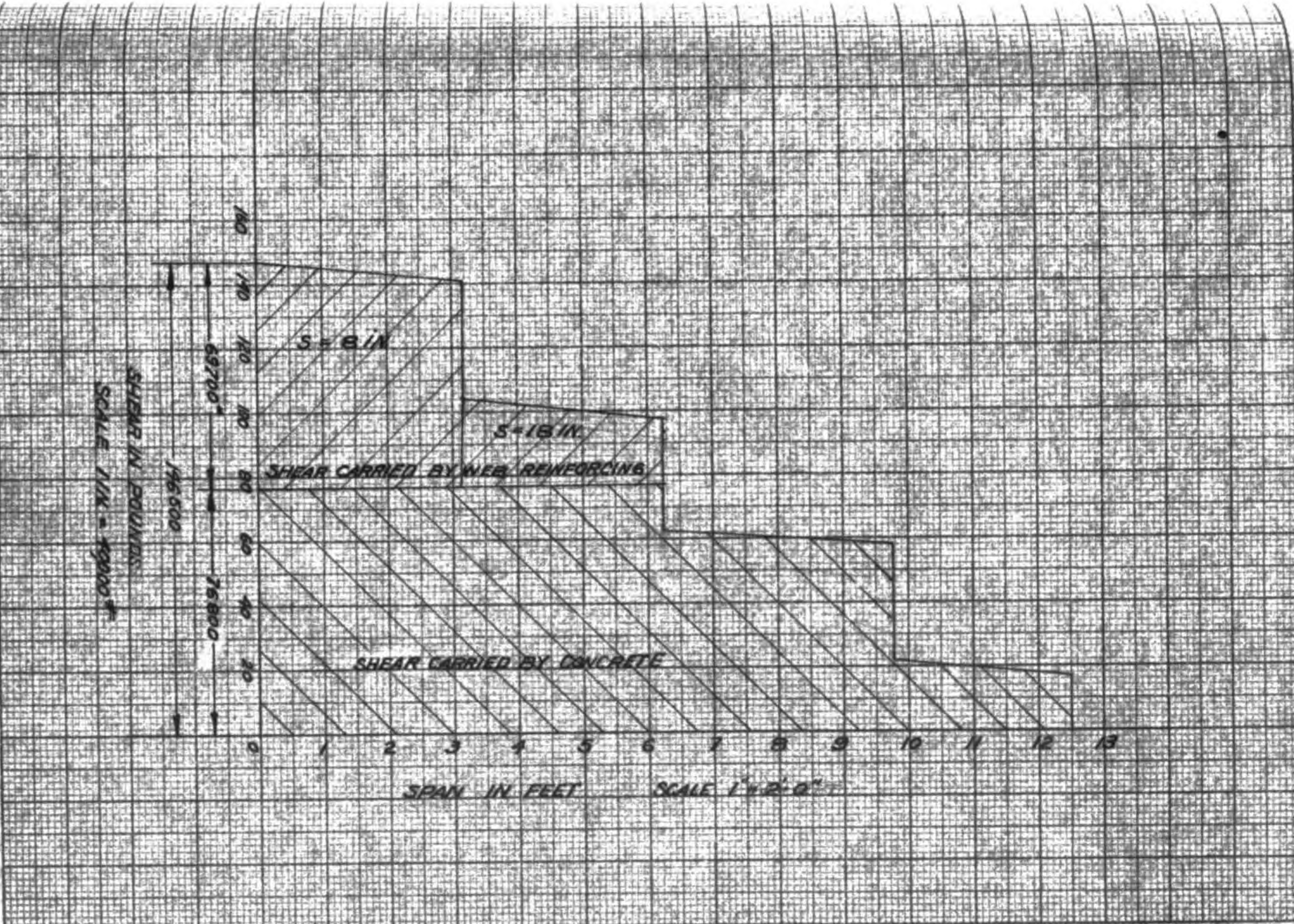
$$V_s = V - V_c = 146.5 \times 1000 - 76800 = 69{,}700\#.$$

Let S = spacing of bars. A_s = cross section area of steel.

Using $\frac{1}{2}$ round bars $S = \frac{A_s f_s J d}{V_s} = \frac{.39 \times 18000 \times .87 \times 92}{69{,}700} = 8.0$ in.

$S = \frac{.39 \times 18000 \times .87 \times 92}{28700} = 19.6$ in. Use 18 in. spacing.

Use 17 $\frac{1}{2}$ ϕ bars MK - B - 400 18" C to C for the remaining length of the beam.


S = 8 IN.
S = 18 IN.
SHEAR CARRIED BY WEB REINFORCING
SHEAR CARRIED BY CONCRETE
160
140
120
100
80
60
40
20
69700#
76800
146500
SHEAR IN POUNDS
SCALE 1 IN. = 40000#
0 1 2 3 4 5 6 7 8 9 10 11 12 13
SPAN IN FEET
SCALE 1" = 2'-0"

Column Design for Pier No. 3

The greatest moment in the column due to transverse loads is at the footing; its magnitude M = 327.0 Kp. Ft.

Working Stresses, J. C. S. Rules

$f_c = 900\#/in^2$ $\quad U = 120$

$f_s = 16000\#/in^2$ $\quad n = 10$

Percent of steel between .005 and .04.

$b^1 = t = 28''$

$M = 327 \times 1000 = 327{,}000$ ft. lb.

N = R + wt. of column (from page 57) wt. col. $= \frac{17}{2} = 8.5$ Kp.

R = Reaction on beam (see page 62) = 171.0 Kp.

$N = 171.0 + 8.5 = 179.5$ Kp. $\times 1000 = 179{,}500\#$.

$$X_0 = \frac{M}{N} = \frac{327{,}000}{189{,}500} = 1.83$$

$$K = \frac{nf_c}{f_s + nf_c} = \frac{10 \times 900}{16000 + 10 \times 900} = .36$$

$$\frac{NX_o}{f_c bt^2} = \frac{179500 \times 1.83}{900 \times 28 \times (28)^2} = .0166$$

$$\frac{t}{x_0} = \frac{28}{1.82} = 15.4$$

From diagram 5 it is evident that the moment due to eccentric loading is small for the length, and the column is designed as a strut.

The minimum longitudinal reinforcement must be bars not less than one inch in diameter and have a total cross sectional

Column Design for Pier No. 3

area of not less than .007 x crossectional area of the column. $A_s = .007 \times (28)^2 = 5.49 \text{ in}^2$

Use 4 - 1" square bars MK - c - 850. Total $A_s = 6.25 \text{ in}^2$.

Use $\frac{1}{4}$ round bars for hoops MK - C - 200 12 in. C to C.

Longitudinal Forces on Pier

Total longitudinal force from the two 30' spans = 2 x 30 x .1 = 6 Kp. (See page 40).

This force is applied 4 feet above the floor, or 18 feet above the footing.

Hence moment due to this load $= \frac{6 \times 18}{2} = 54$ Kp. Ft.

Since this moment is less than the moment due to transverse loading, stresses will be less.

Use same reinforcing in the faces perpendicular to center line of the road.

Design of Footing

Working Stresses

Bars two ways

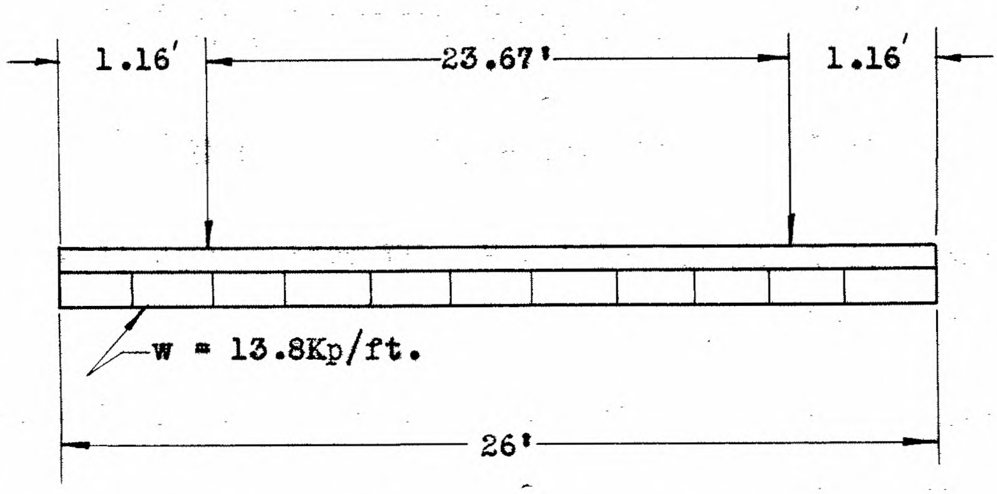
$f_c = 1200\#/in^2$	$n = 10$	$U = .0375f'_c = 112\#/in^2$	No special anchorage $V = .02f'_c = 60\#/in^2$
$f_s = 18000$	$K = 208.3$	$V_p = .06f'_c = 180\#$	Special anchorage required. $V = .03f'_c = 90\#/in^2$

Assuming for footing a slab 3.5 feet deep by 5 feet wide as shown in fig. 14 page 73.

The bearing stress of the soil under the footing in tons per square foot = L.

$$L = \frac{\text{maximum force from columns + wt. of columns + wt. of footing}}{\text{area of footing x 2000}}$$

$$\frac{2(171 + 8.5)1000 + 3.5 \times 5 \times 26 \times 150}{5 \times 26 \times 2000} = 1.64 \frac{\text{tons}}{\text{ft}^2}$$ which is satisfactory.

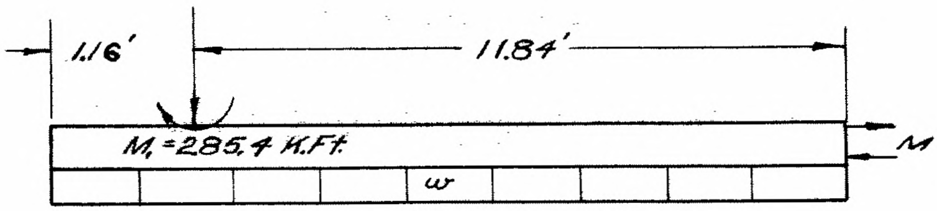


Greatest force from column = R + wt. of column.

R = 171.0 Kp. (page 62) and wt. of column = $\frac{17}{2}$ (page 57).

Design of Footing

$$w = \frac{(171.0 + 8.5)2}{26} = 13.8 \text{ Kp./ft. of length}$$



M_1 = moment from column = 285.4 (see page 62).

$$M = 167.0 \times 11.84 - 285.4 - \frac{13.8 \times (11.83)^2}{2}$$

$$M = 727.0 \text{ Kp. ft.}$$

$$A_s = \frac{M}{f_s JU} = \frac{727.0 \times 1000 \times 12}{18000 \times .87 \times 39.0} = 14.3 \text{ in}^2.$$

$$\Sigma O = \frac{V}{UJd} = \frac{R}{UJd} = \frac{171.1 \times 1000}{112 \times .87 \times 39.0} = 45 \text{ in.}$$

Use 4 - $1\frac{1}{4}$ square bars MK - F - 1050 12 in. C to C through and hooked at both ends.

Use 5 - $1\frac{1}{4}$ square bars MK - F - 1051 12 in. C to C bent at P.I. as shown in fig.14

A_s used = 14.1 ΣO used = 45 in.

Design of Footing

Check for Double Reinforcing

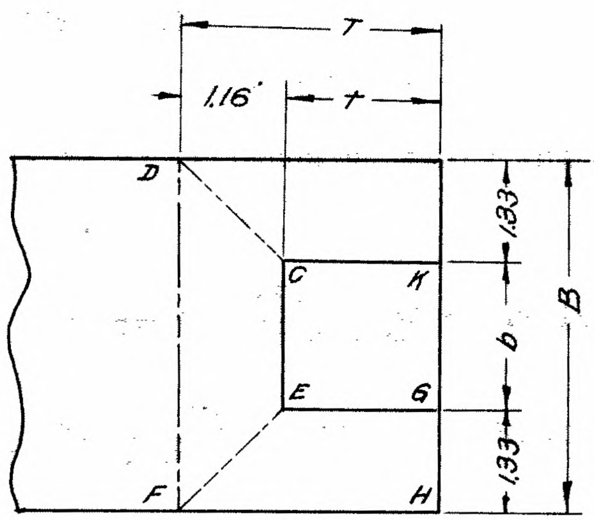
$M = 727.0 \times 1000 \times 12 = 8{,}730{,}000$ in. pounds.

$M_1 = Kbd^2 = 208.3 \times 60(39)^2 = 19{,}000{,}000$ in pounds.

Since $M_1 > M$ compression steel is not required.

Check for Shear

$$v = \frac{V}{bJd} = \frac{171.0 \times 1000}{60 \times .87 \times 39} = 84.0 \ \#/in^2.$$



Assuming that only T length of the footing resists the longitudinal moment, $B = 5'$ $b = 2.33'$; $T = 4.65'$ $t = 2.33'$.

N = total load on footing = 171.0 Kp.

Y = upward thrust on footing per square foot $= \frac{171 \times 1000}{5 \times 4.65}$

$= 7370 \ \#/ft^2.$

P_1 = punching shear along CE = (area of CEFD)Y

$$P_1 = 4.24 \times 7370 = 31400\#$$

Design of Footing

P_2 = punching shear along EG = (area of EGHF)Y

$$P_2 = (2.33 \times 1.33 + \frac{1.16 \times 1.33}{2})Y = 3.9 \times 7370 = 28700\#$$

M_1 = moment along CE = (area of CDEF)(upward thrust)(distance from center of area of CDEF to CE)

$M_1 = 4.24 \times 7370 \times .77$ $\qquad$ $M_1 = 24000'\#$

M_2 = moment along EG = (Area of EGHF)(upward thrust)(distance from center of area of EGHF to EG)

$M_2 = 3.9 \times 7370 \times .7$ $\qquad$ $M_2 = 20200'\#$

$$M_2 = 20200'\#$$

Using the largest value of punching shear and moment to check for d,

$$d = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{24000 \times 12}{208.3 \times 60}} = 4.7 \text{ in.}$$

$$d = \frac{P}{12V_p t} = \frac{31400}{12 \times 180 \times 28} = .52 \text{ in.} \qquad \text{Used for } d = 39 \text{ in.}$$

Check for Tensile Steel

f_t if $> .01f'_c$ or $> 30\#/in^2$ steel must be used.

$$f_t = \frac{6M_1}{bd^2} = \frac{6 \times 24000 \times 12}{28 \times (39)^2} = 54.6 \ \#/in^2 \quad \text{steel is required .}$$

$$f_t = \frac{6M_2}{bd^2} \quad \frac{6 \times 20200 \times 12}{28 \times (39)^2} = 34.3 \ \#/in^2 \quad \text{steel is required.}$$

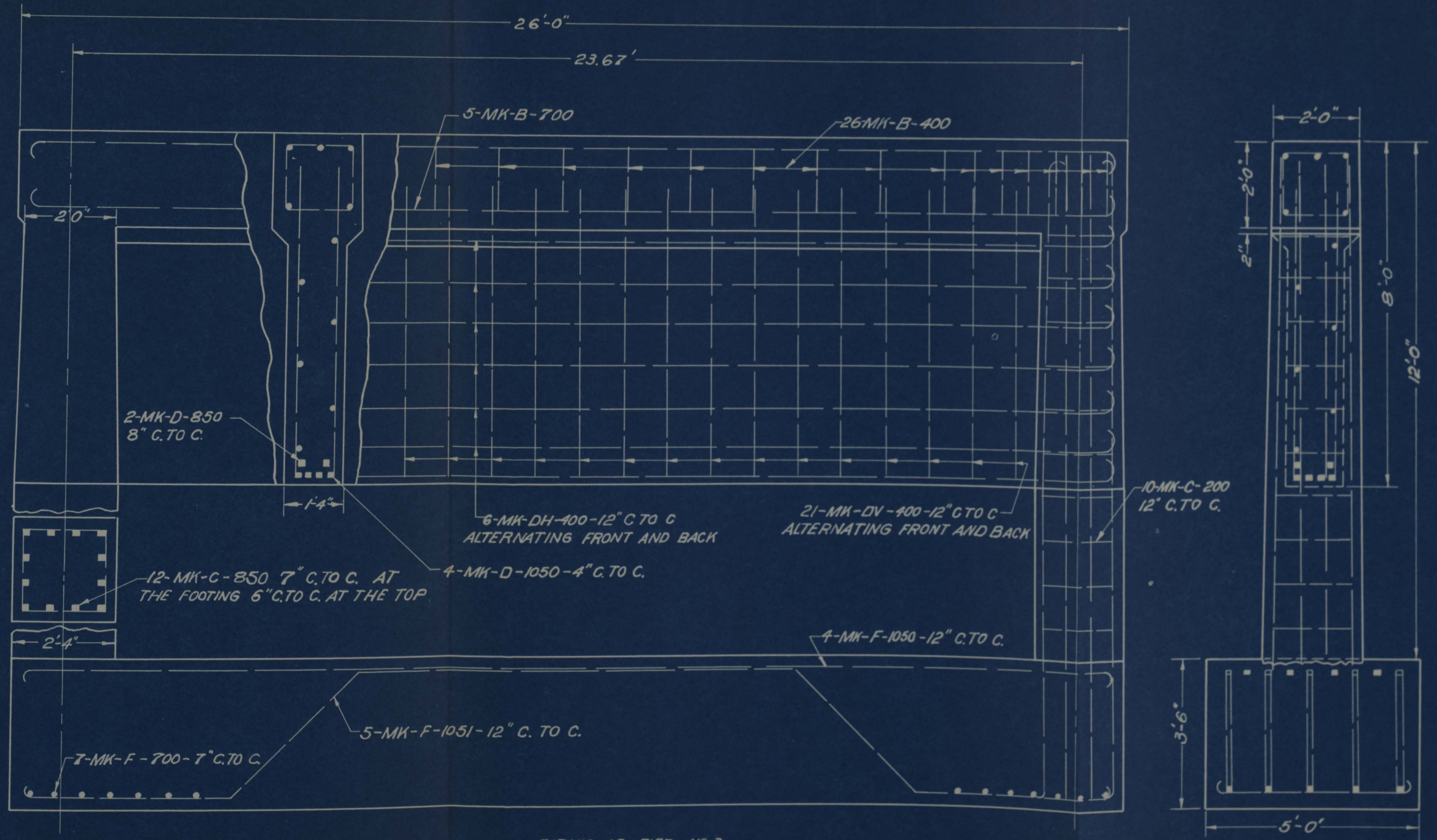
$$A_s = \frac{M_1}{f_s j d} = \frac{24,000 \times 12}{18000 \times .87 \times 39} = .473 \text{ in}^2.$$

Design of Footing

Use 7 - $\frac{1}{2}$ ϕ bars. MK - F - 400 7 in. C to C from each end of footings.

$$A_s = \frac{M_s}{f_s j d} \quad \frac{20200 \times 12}{18000 \times .87 \times 39} = .4 \text{ in}^2.$$

4 - bars MK - F - 1051 are bent down from the top of the footing, as shown in fig. 14 page 73.



DETAILS OF PIER NO.3
FIGURE 14
SCALE $\frac{1}{2}$" = 1'-0"

Fig 14

Design of Abutment

The greatest load on the abutment will occur when the rear wheels of the 15 ton truck are over the abutment in both lanes of roadway.

Maximum load on abutment = LL + DL + IL = 118500#

LL = 24,000 x 2 = 48,000#

DL = 9045 x 7 (page 31) = 63,300#

IL = 24,000 x .3 = 7200#

Weight of abutment shown in figure 15 = 66200#.

Assuming that the slab portion of abutment does not provide any effective bearing area, total force on the bottom of abutment = 184,700#.

Let B = soil pressure on the bottom of abutment in tons/ft^2.

$$B = \frac{\text{total force on the abutment}}{\text{area of the bottom of abutment}}$$

$$B = \frac{184700\#}{4.5 \times 28 \times 2000} = .734 \text{ tons per foot square}$$

Greatest moment in the abutment in plane parallel to the C.L. of the roadway occurs midway between the floor beams.

$M = \frac{wL^2}{8}$ where M = moment per foot of width

L = span or distance between center lines of beams. L = 3.125'

w = pressure of soil in pound per foot of length.

$$w = \frac{118500}{4.5 \times 28} = 940\#/\text{ft}.$$

Design of Abutment

$$M = \frac{940 \times 1(3.125)^2}{8} = 1,150'\#$$

$$A_s = \frac{M}{f_s J d} = \frac{1150 \times 12}{18000 \times .87 \times 19} = .047 \text{ in}^2$$

$$\Sigma 0 = \frac{V}{UJd} = \frac{8950}{120 \times .87 \times 19} = 4.53 \text{ in.}$$

Use 6 - $\frac{1}{2}\phi$ bars MK - A - 400, 10 in. C to C full length of abutment and hooked at both ends.

The greatest moment in the abutment in plane perpendicular to of roadway occurs under the floor beams.

$$M = \frac{wL^2}{2}$$

Where M = moment per foot of width,

W = pressure of soil in pounds per foot of length

L = half of the width of abutment = 2.25'

$$M = \frac{1150 \times (2.25)^2}{2} = 2910'\#$$

$$A_s = \frac{2910 \times 12}{18000 \times .87 \times 19} = .12 \text{ in}^2$$

$$\Sigma 0 = \frac{V}{UJd} = \frac{1468 \times 2.25}{120 \times .87 \times 19} = 1.28 \text{ in.}$$

Use 27 - $\frac{1}{2}$ square bars, MK - A - 450 $12\frac{1}{2}$ in. C to C.

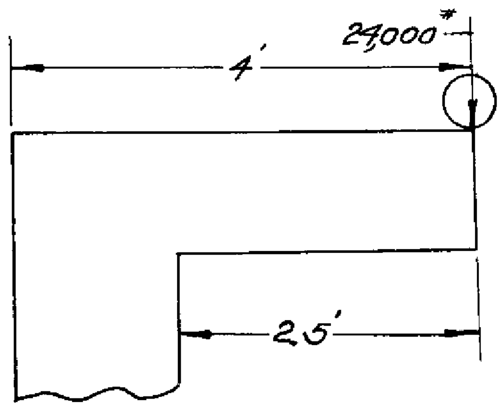
Maximum moment in the slab portion of the abutment occurs at the vertical wall when the wheels of the truck are on the edge of the slab as shown.

Design of Abutment

Total maximum force = wheel loads

+ IL = 24000 x 2 + 24000 x 2 x .3

= 62400#.



Force per foot of width =

$$\frac{62,400}{\text{width of roadway}} = 2600 \text{ \#/foot.}$$

Moment at the vertical wall M =

2600 x 2.5 = 6500'#

$$A_s = \frac{6500 \times 12}{18000 \times .87 \times 9} = .55 \text{ in}^2.$$

$$\Sigma 0 = \frac{2600}{120 \times .87 \times 9} = 2.78 \text{ in.}$$

Use 54 - $\frac{1}{2}\phi$ bars, MK - AS - 401 6 in. C to C bent as shown in fig.15

Steel in slab part of abutment transverse to roadway is temperature steel. Use 5 bars in each top and bottom face MK - AS - 401, 9 in. C to C.

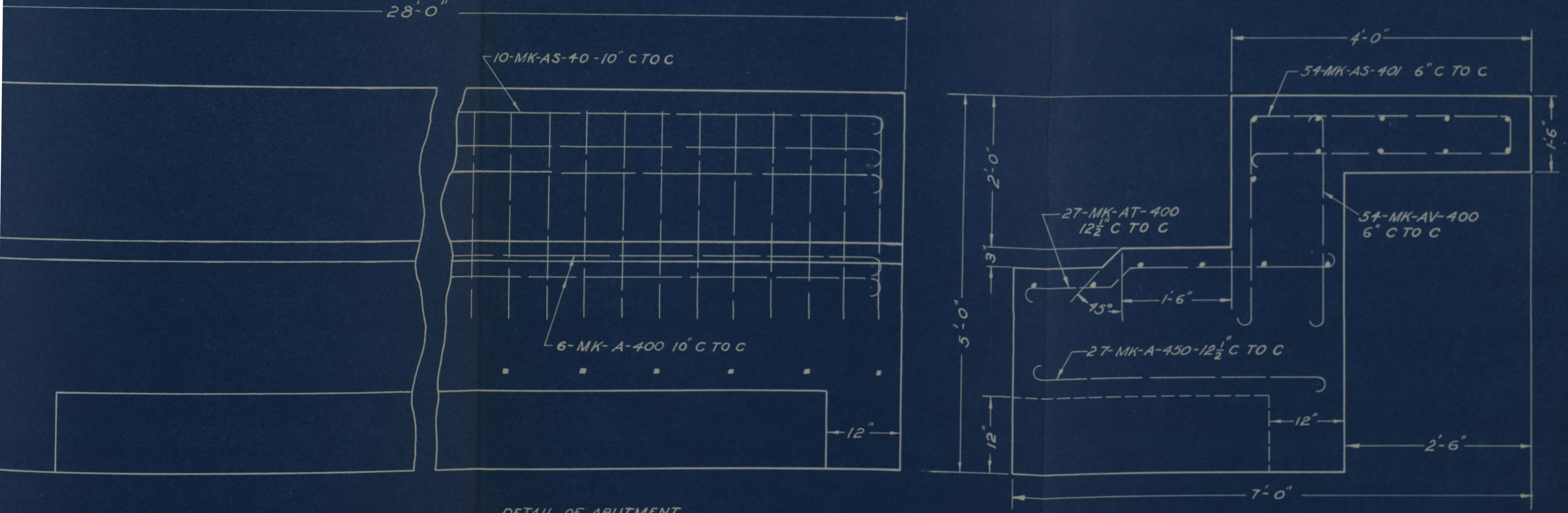
Moment in the vertical wall due to the same loading,

M = 2600 x 3.25 = 8450'#

$$A_s = \frac{M}{f_s jd} \quad \frac{8450 \times 12}{18000 \times .87 \times 15} = .433 \text{ in}^2$$

Use 54 - $\frac{1}{2}\phi$ bars, MK - AV - 400,

6 in. C to C bent and hooked as shown in fig.15

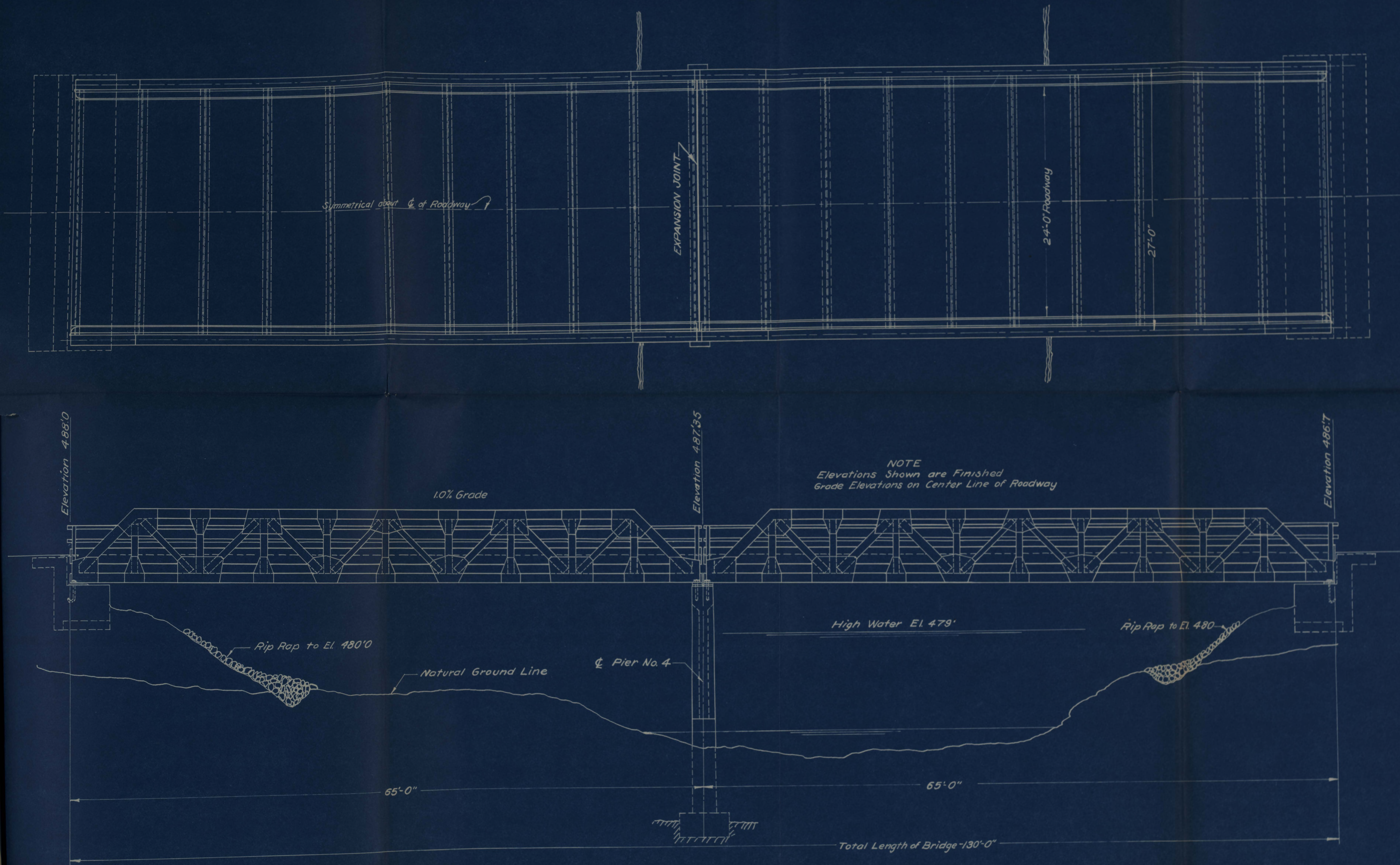


DETAIL OF ABUTMENT
FIGURE 15
SCALE 1"=1'-0"

Fig 15

PART III

LOW WARREN TYPE THROUGH TRUSS BRIDGE DESIGN



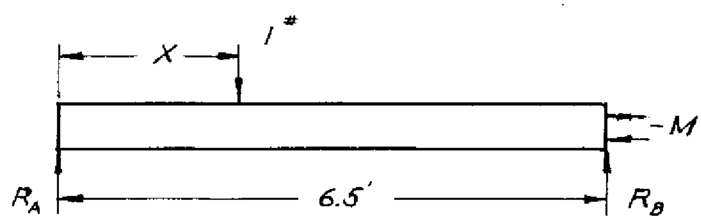
LOW WARREN TYPE THROUGH TRUSS
Scale 3/16" = 1'-0"

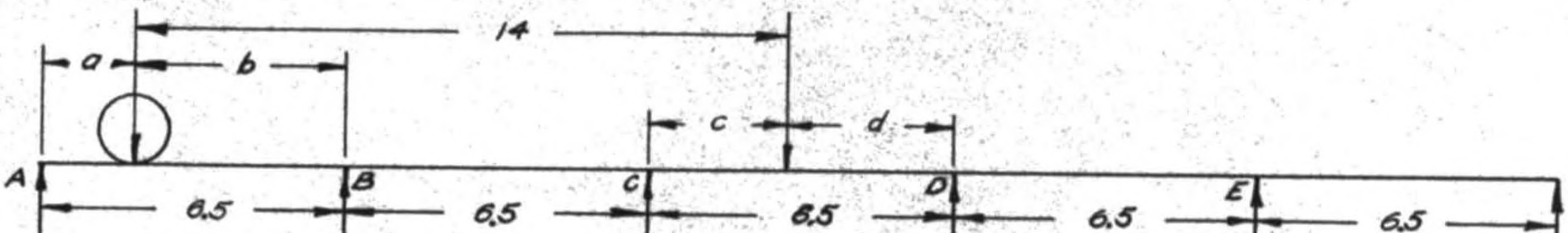
FIGURE 16

Design of Concrete Slab for the Through Low Warren Type Truss

Each truss has eleven floor beams which support the slab. The slab is then designed as a beam continuous over eleven supports and is considered free at the end supports. Since the slab is considered free at the ends, the maximum negative moment will be over the first intermediate support and the maximum positive moment will occur in the first panel. To locate the position of the truck wheels producing maximum negative moment, loads were placed in position relative to the rear and front wheels and proportional to the rear and front wheel loads. The negative moment over the first intermediate support was found by moment distribution method for seven positions of the wheel loads. A curve with the moment values as ordinate and the positions of wheels on the span as abscissa was plotted.

To determine the position of the truck wheels producing maximum positive moment, the first span of the slab is shown as a free body. A unit load is placed in the same positions as in determining the maximum negative moment, and the value of R_a is evaluated.





A	B	B	C	C	D	D	E	E		
+.96	−.428	0	0	+.218	−.1867	0	0	0	0	0
−.96	+.214	+.214	−.109	−.109	+.0934	+.0934	0	0	0	0
+.107	−.48	−.045	+.107	+.0466	−.0545	0	+.0466	0	0	0
−.107	+.267	+.267	−.0768	−.0768	+.0273	+.0273	−.0233	−.0233	0	0
+.133	−.0535	−.038	+.1335	+.0136	−.0384	−.0117	+.0136	0		
−.133	+.0460	+.0460	−.0736	−.0736	+.025	+.025	−.0068	−.0068		
	−.4346	+.4346	−.0189	+.0189	−.134	+.134				

$M_{FAB} = Pa\left(\frac{b}{6.5}\right)^2$ WHEN, $a=2, b=4.5$ $c=3$ $d=3.5$

$M_{FAB} = 1 \times 2\left(\frac{4.5}{6.5}\right)^2 = .96$

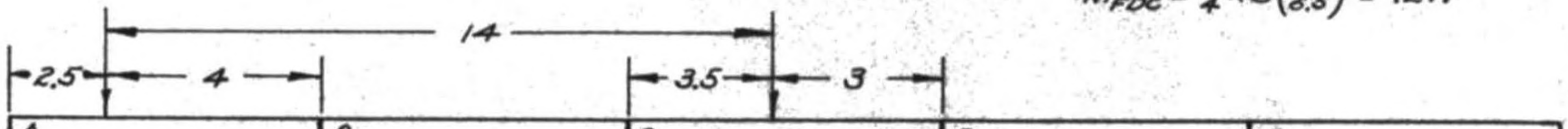
$M_{FBA} = Pb\left(\frac{a}{L}\right)^2 = 1 \times 4.5\left(\frac{2}{6.5}\right)^2 = .428$

$M_{FCD} = Pc\left(\frac{d}{6.5}\right)^2 = \frac{1}{4} \times 3\left(\frac{3.5}{6.5}\right)^2 = .218$

$M_{FDC} = Pd\left(\frac{c}{6.5}\right)^2 = \frac{1}{4} \times 3.5\left(\frac{3}{6.5}\right)^2 = .1867$

$M_{FAB} = 1 \times 2.5\left(\frac{4}{6.5}\right)^2 = .948$ $\qquad M_{FBA} = 1 \times 4\left(\frac{2.5}{6.5}\right)^2 = .592$ $\qquad M_{FCD} = \frac{1}{4} \times 3.5\left(\frac{3}{6.5}\right)^2 = .1866$

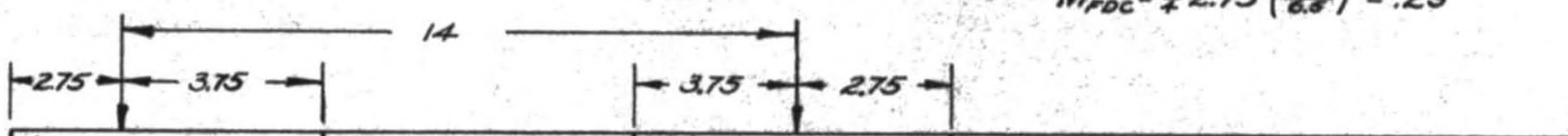
$M_{FDC} = \frac{1}{4} \times 3\left(\frac{3.5}{6.5}\right)^2 = .217$



A		B		C		D		E	
+.948	-.592	0	0	+.1865	-.217	0	0	0	0
-.948	+.296	+.296	-.0932	-.0932	+.1085	+.1085	0	0	
+.148	-.474	-.0466	+.148	+.0543	-.0466	0	+.0542		
-.148	+.2603	+.2603	-.1011	-.1011	+.0233	+.0233	-.0271	-.0271	
+.130	-.074	-.06	+.130	+.0116					
-.130	+.067	+.067	-.0708	-.0708					
	-.517	+.517	+.0129	-.0128					

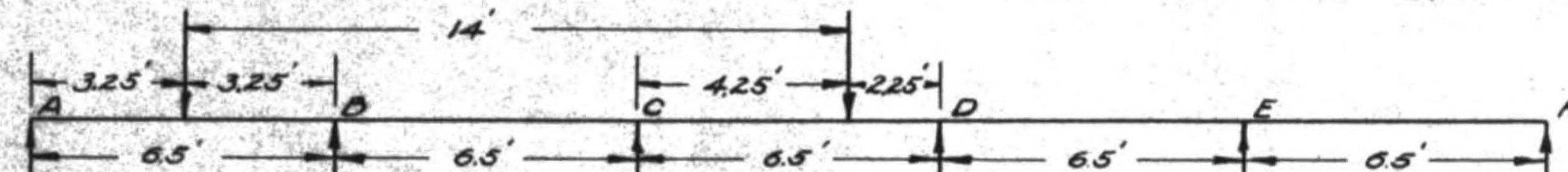
$M_{FAB} = 1 \times 2.75\left(\frac{3.75}{6.5}\right)^2 = .92$ $\qquad M_{FBA} = 1 \times 3.75\left(\frac{2.75}{6.5}\right)^2 = .671$ $\qquad M_{FCD} = \frac{1}{4} \times 3.75\left(\frac{2.75}{6.5}\right)^2 = .168$

$M_{FDC} = \frac{1}{4}\, 2.75\left(\frac{3.75}{6.5}\right)^2 = .23$



A		B		C		D		E
+.92	-.671	0	0	+.168	-.23		0	
-.92	+.3355	+.3355	-.084	-.084	+.115	+.115	0	
+.1678	-.46	-.042	+.16775	+.0575	-.042	0	0	
-.1678	+.251	+.251	-.1126	-.1126	+.021	+.021		
+.125	-.0838	-.0563	+.12					
-.125	+.07	+.07						
	-.558	+.558						

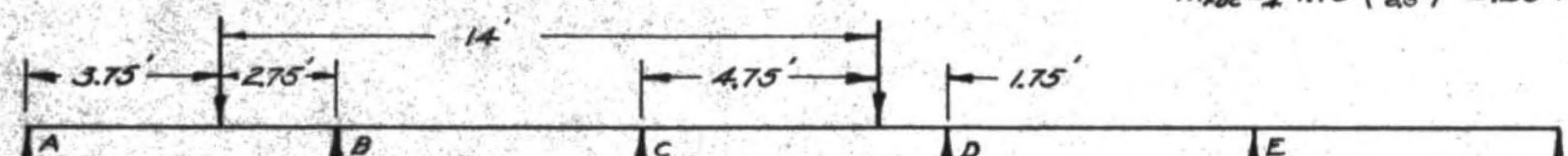
$M_{FAB} = 1 \times 3.25\left(\frac{3.25}{6.5}\right)^2 = .814 = M_{FBA}$ $\quad M_{FCD} = \frac{1}{4}\, 4.25\left(\frac{2.25}{6.5}\right)^2 = .128$ $\quad M_{FDC} = \frac{1}{4}\, 2.25\left(\frac{4.25}{6.5}\right)^2 = .242$



A	B	B	C	C	D	D	E	E	F
+.814	−.814	0	0	+.128	−.242	0	0	0	0
−.814	+.407	+.407	−.064	−.064	+.121	+.121	0	0	0
+.2035	−.407	−.032	+.2035	+.0605	−.032	0	+.0605		
−.2035	+.2195	+.2195	−.1320	−.1320	+.016	+.016	−.0302	−.0302	
+.1098	−.1018	−.066	+.10975	+.008	−.0660				
−.1098	+.0839	+.0839	−.0589	−.0588					
0	−.6124	+.6124	+.0584	−.0584					

$M_{FAB} = 1 \times 3.75\left(\frac{2.75}{6.5}\right)^2 = .673$ $\quad M_{FBA} = 1 \times 2.75\left(\frac{3.75}{6.5}\right)^2 = .915$ $\quad M_{FCD} = \frac{1}{4}\, 4.75\left(\frac{1.75}{6.5}\right)^2 = .0858$

$M_{FDC} = \frac{1}{4}\, 1.75\left(\frac{4.75}{6.5}\right)^2 = .234$



A	B	B	C	C	D	D	E	E	
+.673	−.915	0	0	+.0858	−.234	0	0	0	0
−.673	+.4575	+.4575	−.0429	−.0429	+.117	+.117			
+.2288	−.3365	−.0214	+.2288	+.0585	−.0214	0	+.0585		
−.2288	+.179	+.179	−.1436	−.1436	+.0108	+.0108	−.0292		
+.0895	−.1144	−.0718	+.0895						
−.0895	+.0931	+.0931							
0	−.6363	+.636							

When $x = 2'$

Value of M is obtained from the moment distribution (page 81)

$$M = -.4346'\#$$

$$\Sigma M \text{ about } R_B = 6.5R_A + .4346 - 4.5 \times 1 = 0$$

$$R_A = .625\#$$

Positive moment at a section 2 feet from R_A

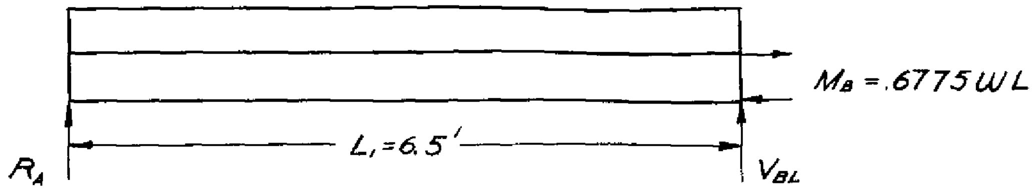
$$M = R_A \times 2 \quad \text{or } M = .625 \times 2 = 1.25'\#$$

Value of x ft.	Value of M coef.	Value of $R_A\#$	Value of $M'\#$
2	.4346	.625	1.25
2.5	.517	.536	1.34
2.75	.558	.491	1.35
3.25	.612	.405	1.32
3.75	.636	.325	1.22
4.5	.607	.214	.965
5	.534	.0717	.358

The maximum positive and negative moment is obtained by multiplying the moment due to unit load by the actual load from wheels.

Slab Design Cont'd.

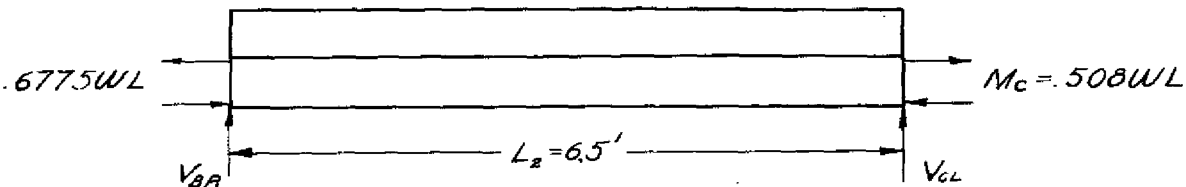
To determine the position of maximum moment due to DL the first and second spans of the slab are shown as a free body, reaction R_A and R_B evaluated, and shear diagram drawn.



Value of M_B was obtained from the moment distribution for uniform load of encased I beam design.

$$\leq M_{V_{BL}} = R_A L + .6775 wL_1 - \frac{wL^2}{2} = 0 \qquad R_A = 2.57w$$

$$\leq M_{R_A} = V_{BL}L - .6775wL - \frac{wL^2}{2} \qquad V_{BL} = 3.93w$$



$$\leq M \text{ about } V_{CL} = V_{BR}L + .508wL - .6775wL - \frac{wL^2}{2} = 0 \qquad V_{BR} = 3.42wL$$

$$R_B = V_{BL} + V_{BR} \qquad R_B = 7.35w$$

Let E = effective width of slab in feet for one wheel load.

$E = \frac{2}{3}L$ (concrete design notes) $\qquad E = \frac{2 \times 6.5}{3} = 4.33$ ft.

Moment per foot of width of slab over

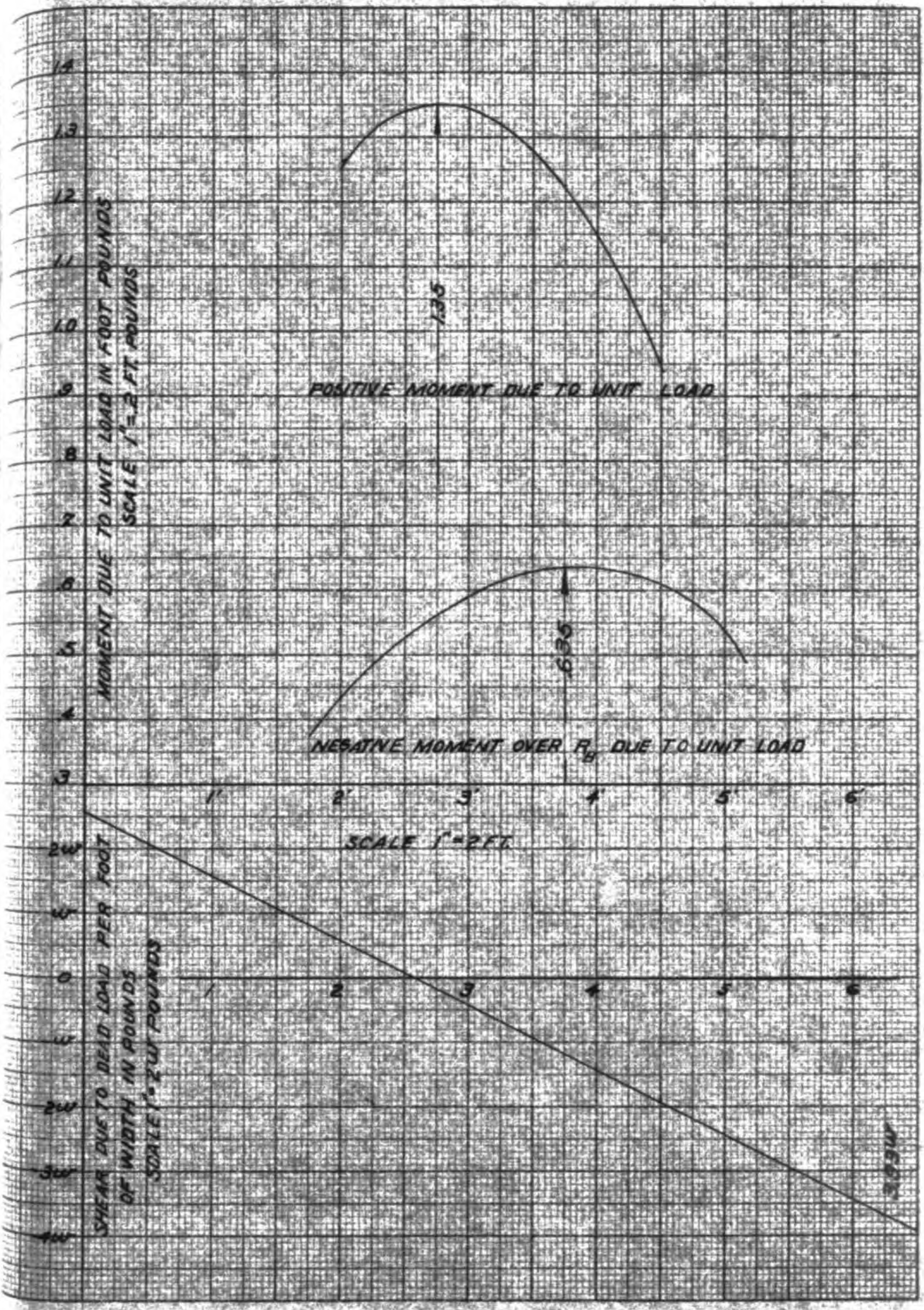
$$R_B = \frac{\text{(moment due to unit load) x (load per wheel)}}{\text{effective width}}$$

Negative M due to LL = $\frac{.635 \times 12000}{4.33} = 1760$ ft.#

Negative M due to IL = 1760 x 3 = 528 ft.#

Negative M due to DL = .6775wL = 4.4w ft.#

Total negative M = 2288 + 4.4w



MOMENT DUE TO UNIT LOAD IN FOOT POUNDS
SCALE 1" = .2 FT. POUNDS
1.4
1.3
1.2
1.1
1.0
.9
.8
.7
.6
.5
.4
.3
POSITIVE MOMENT DUE TO UNIT LOAD
1.35
NEGATIVE MOMENT OVER R_B DUE TO UNIT LOAD
.635
1' 2' 3' 4' 5' 6'
SCALE 1" = 2 FT.
SHEAR DUE TO DEAD LOAD PER FOOT
OF WIDTH IN POUNDS
SCALE 1" = 2W POUNDS
2W
W
0
1 2 3 4 5 6
-W
-2W
-3W
-4W
3.83W

Slab Design Cont'd.

Positive LLM = $1.35 \times \frac{12000}{4.33}$ = 3740'#

Positive IM = 3740 x .3 = 1122'#

Positive DLM = $(2.57w)2.6 - (2.6)^2 \frac{w}{2} = 3.03w$

Positive total moment = 4862 + 3.03w

Assume d = 7.5 in. d^1 = 1.5 in. Total depth of slab = 7.5 + 1.5 = 9"

Weight of slab per square foot = weight of conc. + weight of covering.

Weight of slab per square foot = 9 x 12.5 + 14.5 = 127#/ft^2

Total positive M = 4862 + 3.03 x 127 = 5247'#, say 5500'#.

Total negative M = 2288 + 4.4w

M = 2288 + 4.4 x 127 = 2846'#

Working stresses are same as in the first design.

E for shear = $\frac{3.33 \times d}{12}$ $\frac{3.33 \times 7.5}{12}$ = 2.08 ft.

Shear over R_B = $\frac{12000}{2.08} + V_{BL}$ $\quad V_{R_B}$ = 5770 + 3.93 x 127

Shear over R_B = 6270

Shear at center = .5 x 5770 = 2885#

Shear at PI = $\frac{6270 + 2885}{2}$ = 4577#

$$d = \sqrt{\frac{M}{bK}} = \sqrt{\frac{12 \times 6270}{12 \times 208.3}} = 5.48 \text{ in.}$$

$$d = \frac{V}{bJv} = \frac{6270}{12 \times .87 \times 90} = 6.66 \text{ in.}$$ Use d = 7.5

Slab Design Cont'd.

Special anchorage is required, therefore $\frac{7}{12}$ of steel required at center must go through over the support at bottom.

$$A_s \text{ top} = \frac{-M}{f_s Jd} = \frac{2846 \times 12}{18000 \times .87 \times 7.5} = .29 \text{ in}^2.$$

$$A_s \text{ bottom} = \frac{+M}{f_s Jd} = \frac{5500 \times 12}{18000 \times .87 \times 7.5} = .562 \text{ in}^2.$$

Using deformed bars, value of u must not exceed $.05f_c = 150\#/\text{in}^2$.

$$\Sigma 0 \text{ at top} = \frac{V}{uJd} = \frac{6270}{150 \times .87 \times 7.5} = 6.48 \text{ in.}$$

$$\Sigma 0 \text{ at bottom} = \frac{2885}{150 \times .87 \times 7.5} = 2.95 \text{ in.}$$

$$\Sigma 0 \text{ at PI} = \frac{4577}{150 \times .87 \times 7.5} = 4.68$$

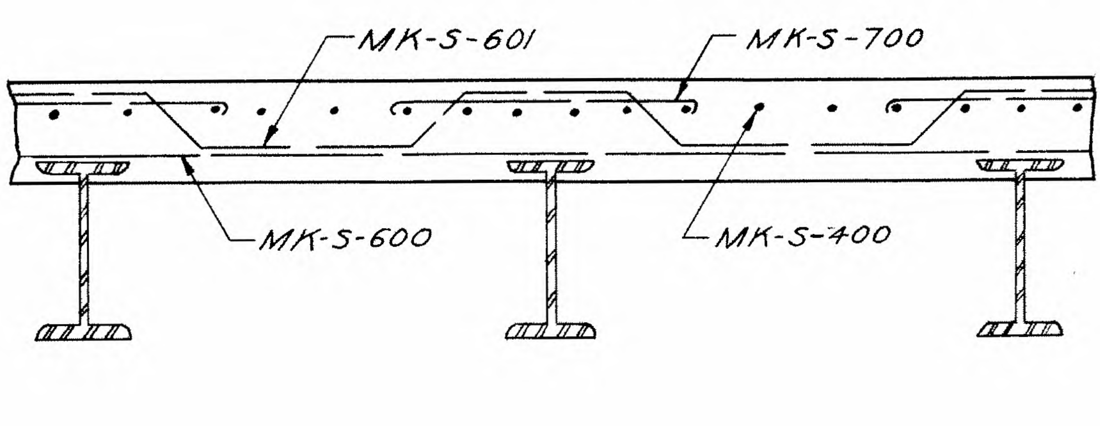
Bond at support governs

Use $\frac{3}{4}$" ø bars MK - S - 600 12 in. C to C straight through at bottom.

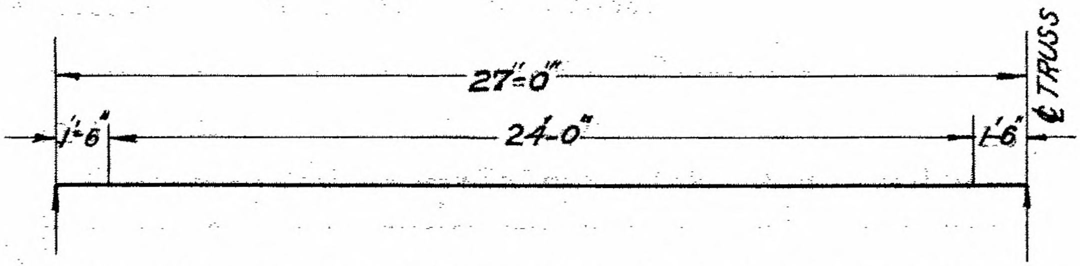
Use $\frac{3}{4}$" ø bars MK - S - 601 12 in. C to C at bottom, bent up at PI over the supports ect.

Use bars $\frac{7}{8}$ ø MK - S - 700 12 in. C to C over the support and hooked beyond PI.

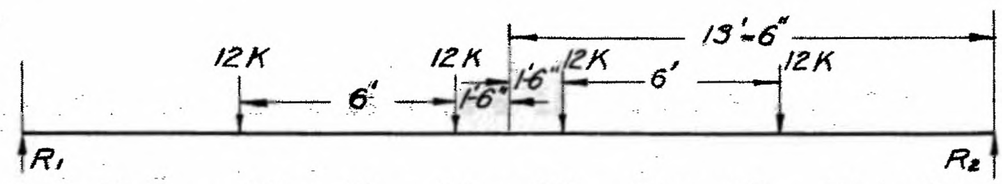
For transverse reinforcement, use $\frac{1}{2}$ ø bars MK - S - 400 12 in. C to C full length of span.



Floor Beam Design



For maximum moment at center place loads as shown



$$\Sigma M_{R_1} = 27\ R_2 - 12 \times 21 - 12 \times 15 - 12 \times 12 - 12 \times 6 = 0$$

$$R_1 = R_2 = 24\ K.$$

LLM at the center = 24 x 13.5 - 12 x 7.5 - 12 x 1.5 = 216K.Ft.

Imp. M = 216 x .3 = 64.8 K.Ft.

Weight of slab/ft. = 127 x 6.5 = 825#/ft.

Assume weight of beam 104#/ft.

Total weight per foot = .93 Kp.

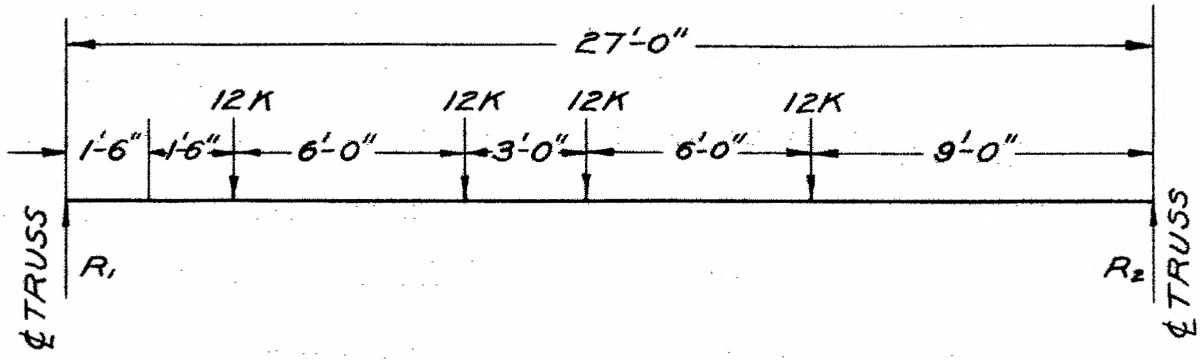
DLM at center = $\frac{.93(27)^2}{8}$ = 84.8 K. ft.

Total M = (216 + 64.8 + 84.8)12 x 1000 = 4,380,000 in. lb.

Section Modulus = $\frac{4380000}{18000}$ = 244.0

Section Modulus for 27"98#I beam = $\frac{I}{C} = \frac{3446.5}{13.5}$ = 256.

For maximum shear place LL as shown.



$\Sigma M_{R_2} = 12 \times 24 + 12 \times 18 + 12 \times 15 + 12 \times 9 - R_1 27 = 0$

$R_1 = 29.3$ Kp.

R_1 = Reaction due to LL = 29.35K.

Assuming weight of curb per panel = .7 Kp.,

DL $R_1 = \frac{wL}{2}$ + wt. of curb + wt. of beam = $\frac{95 \times 27}{2}$ +

$\frac{.098 \times 27}{2} + .7$

Where R_1 = reaction to DL = 13.4, R_1 due to impact = 29.35 x .3 = 8.8K.

Total reaction or total shear at the support = 29.35 + 13.4 + 8.8 = 51.5 Kp.

Shearing area required = $\frac{51.5 \times 1000}{11{,}000}$

Shearing area required = 4.65 sq. in.

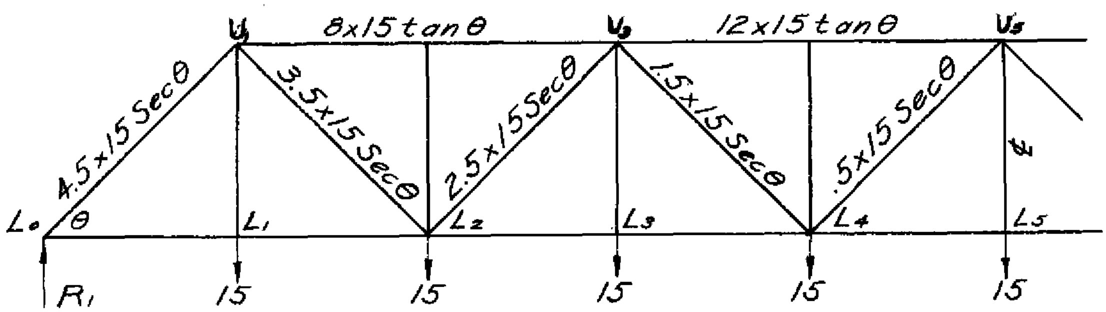
Flexure governs

Use 27 - 10 in. flange 98# CIB.

Dead Load on One Truss

Weight of slab = 127 x 12.5 x 65	= 103,000#
Assume weight of curb	= 7,000#
Assume weight of structural metal per truss	= 40,000#
Total weight per truss	= 150,000#

Weight per panel = $\frac{150000}{10 \times 1000}$ = 15.0 Kp.



R_1 = 4.5 x 15 = 67.5K

Sec = 1.41 Tan = 1

Stresses due to DL

L_0U_1 = 4.5 x 15 x 1.41 = 95.4K.

U_1L_2 = 3.5 x 15 x 1.41 = 74.2K.

L_2U_3 = 2.5 x 15 x 1.41 = 53.0K.

U_3L_4 = 1.5 x 15 x 1.41 = 31.8K.

L_4U_5 = .5 x 15 x 1.41 = 10.6K.

U_1U_3 = 8 x 15 = 120.0K.

U_3U_5 = 12 x 15 = -180.0K.

L_0L_2 = 4.5 x 15 = +67.5K.

L_2L_4 = 10.5 x 15 = +157.5K

L_4L_6 = 12.5 x 15 = +187.5K.

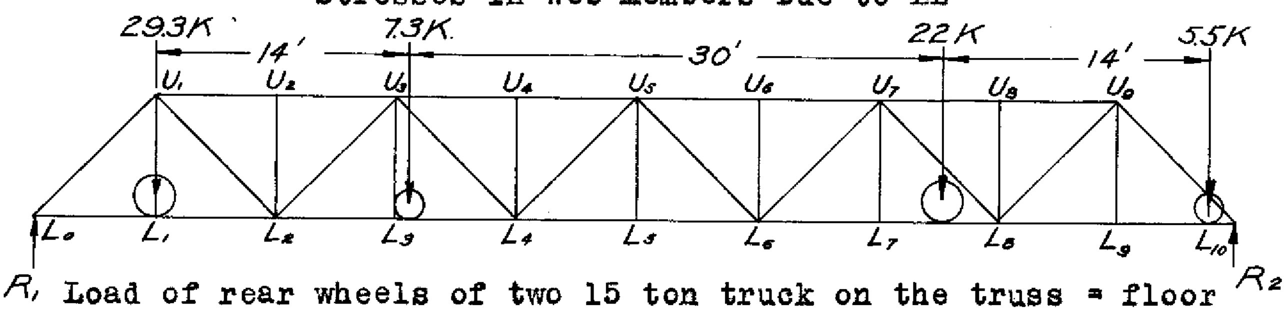
Loading for Maximum Stresses in Web Member Due to LL

Maximum stresses in web members due to LL are obtained by placing the rear wheel of the 15 ton truck at the first panel point and as many other trucks on the span, a specified distance apart, as span will permit; then moving the wheels up one panel length but not allowing any more trucks to come on the span, because that would tend to reduce the shear at the section. (See page94)

Loading for Maximum Stresses in Chord Members Due to LL

Maximum stresses in chord members are obtained by loading same as for web member, but other trucks are allowed to come on the truss within specified distance. (See page 95)

Stresses in Web Members Due to LL



Load of rear wheels of two 15 ton truck on the truss = floor beam reaction when the beam is loaded for maximum LL shear, (See page 91) = 29.3 Kp.

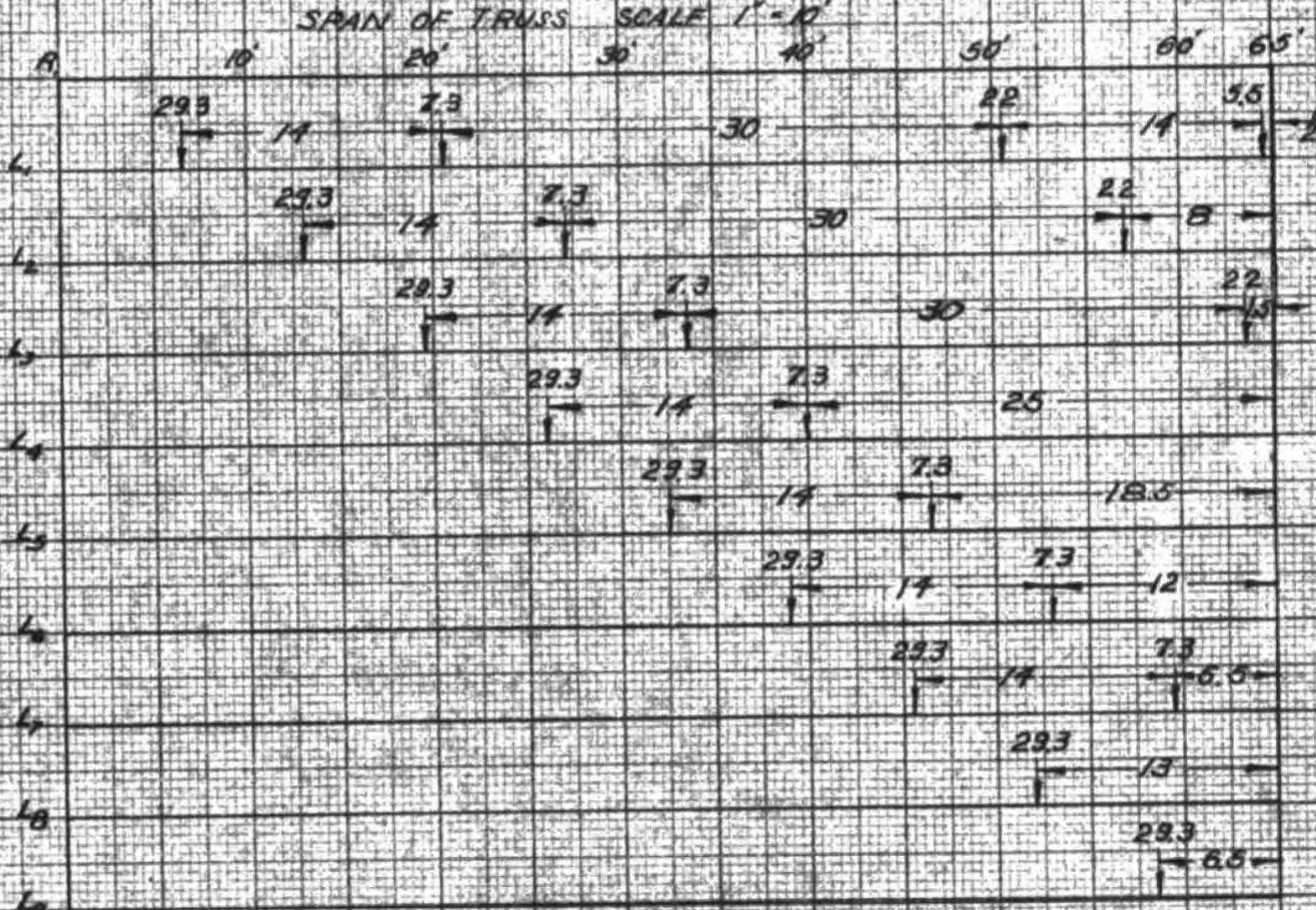
Load of the front wheel of 15 ton truck = $\frac{1}{4}$(rear wheel R) = $\frac{1}{4}$ x 29.3 - 7.3 Kp.

Load of rear wheel concentration of preceding truck ($11\frac{1}{4}$ ton truck) = $\frac{3}{4}$ x 29.2 - 22 Kp.

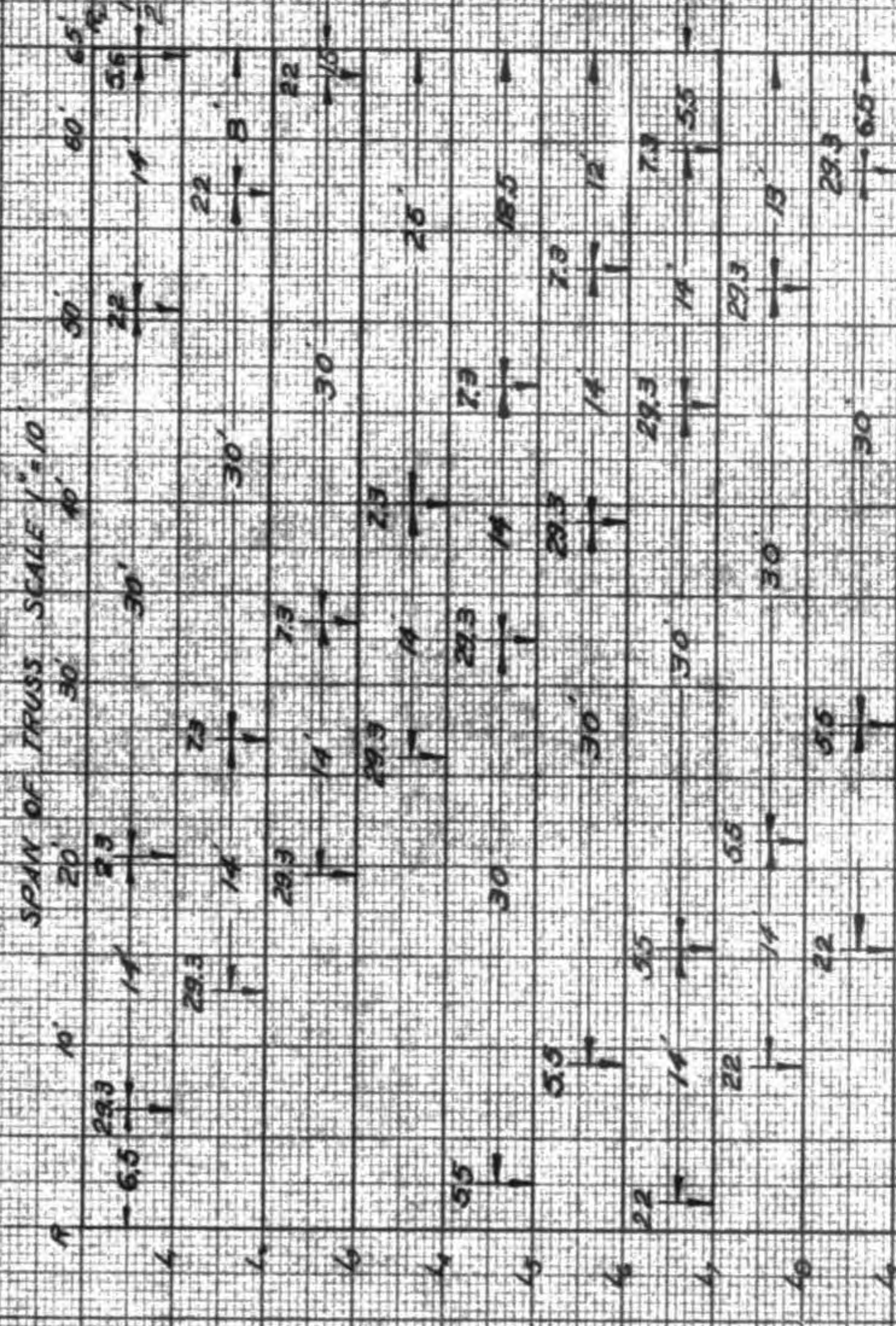
Load of the front wheel of $11\frac{1}{4}$ ton truck = $\frac{1}{4}$(22) = 5.5K

LL LOADING FOR STRESSES IN WEB MEMBERS

SPAN OF TRUSS SCALE 1" = 10'



L.L. LOADING FOR STRESSES IN CHORD MEMBERS



Stresses in Web Members Due to LL

Wheel at L_1

$\Sigma M_{R_2} = 5.5 \times .5 + 22 \times 14.5 + 7.3 \times 44.5 + 29.3 \times 58.5$

$- 65R_1 = 0$

$R_1 = 36.6K$ Stress in $L_0U_1 = \frac{36.6}{.707} = -51.8$ Kp.

Wheel at L_2

$\Sigma M_{R_2} = 22 \times 8 + 7.3 \times 38 + 29.3 \times 52 - 65R_1 = 0$

$R_1 = 30.4$ Stress in $U_1L_2 = \frac{30.4}{.707} = +43$ Kp.

Wheel at L_3

$\Sigma M_{R_2} = 22 \times 1.5 + 7.3 \times 31.5 + 29.3 \times 45.5 - 65R_1 = 0$

$R_1 = 24.5$ Stress in $L_2U_3 = -34.7$ Kp.

Wheel at L_4

$\Sigma M_{R_2} = 7.3 \times 25 + 29.3 \times 39 - 65R_1 = 0$

$R_1 = 20.4$ K. Stress in $U_3L_4 = \frac{20.4}{.707} = +28.9K$

Wheel at L_5.

$\Sigma M_{R_2} = 7.3 \times 18.5 + 29.3 \times 32.5 - 65R_1 = 0$

$R_1 = 16.7$ Stress in $L_4U_5 = \frac{16.9}{.707} = -23.6K$

Wheel at L_6

$\Sigma M_{R_2} = 7.3 \times 12 + 29.3 \times 26 - 65R_1 = 0$

$R_1 = 13.1$ Stress in $U_5L_6 = \frac{13}{.707} = +18.5$

Wheel at L_7

$\Sigma M_{R_2} = 7.3 \times 5.5 + 29.3 \times 19.5 - 65R_1 = 0$

$R_1 = 9.4$ Stress in $L_6U_7 = 13.3$ Kp.

Wheel at L_8

$\Sigma M_{R_2} = 29.3 \times 13 - 65R_1 = 0$ $R_1 = 5.86$ Stress in $U_1L_8 = 8.27$ Kp.

Wheel at L_9

$\Sigma M_{R_2} = 29.3 \times 6.5 - 65R_1 = 0$ $R_1 = 2.92$ Stress in $L_8U_9 = 4.14$ Kp.

Stresses in Chord Members due to LL

By taking the sum. of moments about R_2, R_1 is obtained for loading shown on page 93

Rear wheel of the 15 ton truck at L_1 $R_1 = 36.3$ K

L_2 $R_1 = 30.2$ K

See page 95

L_3 $R_1 = 24.3$ K

L_4 $R_1 = 20.4$ K

$$R_1 = \frac{7.3 \times 18.5 + 29.3 \times 32.5 + 5.5 \times 62.5}{65}$$ L_5 $R_1 = 22.0$ K

$$R_1 = \frac{7.3 \times 12 + 29.3 \times 26 + 5.5 \times 56.}{65}$$ L_6 $R_1 = 17.8$ K

$$R_1 = \frac{7.3 \times 5.5 + 29.3 \times 19.5 + 5.5 \times 49.5 + 22 \times 63.5}{65}$$

L_7 $R_1 = 35.1$ K

$$R_1 = \frac{29.3 \times 13 + 5.5 \times 43 + 22 \times 57}{65}$$ L_8 $R_1 = 28.8$ K

$$R_1 = \frac{29.3 \times 6.5 + 5.5 \times 36.5 + 22 \times 50.5}{65}$$ L_9 $R_1 = 23.1$ K

By the method of sections, stresses in the top and bottom chord were determined, stresses being compression in the top chord and tension in the bottom chord.

Let L = Panel length = 6.5 ft.

Let H = Height of truss = 6.5 ft.

Wheel At	R_1 in Kps.	Stress in chord members due to LL	
L_1	36.6	$L_0L_2 = \frac{R\ L}{H} = \frac{36.3 \times 6.5}{6.5}$	$= 36.6K$
L_2	30.4	$U_1U_3 = \frac{R\ 2L}{H} = 30.2 \times 2$	$= 60.8K$
L_3	24.5	$L_2L_4 = \frac{R\ 3L}{H} = 24.3 \times 3$	$= 73.5K$
L_4	20.4	$U_3U_5 = \frac{R\ 4L}{H} = 20.4 \times 4$	$= 81.6K$
L_5	22.0	$L_4L_6 = 22 \times 5 - \frac{5.5 \times 30}{6.5}$	$= 84.6K$
L_6	17.8	$U_5U_7 = 17.8 \times 6 - \frac{5.5 \times 30}{6.5}$	$= 81.4K$
L_7	35.1	$L_6L_8 = 35.0 \times 7 - \frac{5.5 \times 30 - 22 \times 44}{6.5}$	$= 70.7K$
L_8	28.8	$U_7U_9 = 28.8 \times 8 - \frac{5.5 \times 30 - 22 \times 44}{6.5}$	$= 56.1K$
L_9	23.1	$L_8L_{10} = 22.9 \times 9 - \frac{5.5 \times 30 - 22 \times 44}{6.5}$	$= 33.2K$

Impact Allowance

Live load stresses for spans over 45 feet in length are increased by a fraction of the live load given by the following formula: $I = \frac{50}{L + 125}$

Where I = impact fraction, L = length of span in feet.

$I = \frac{50}{65 + 125} = .263$ Then IL = .263 x LL.

Tabulation of Stresses in Web Members

Loading	L_0U_1	U_1L_2	L_2U_3	U_3L_4	L_4U_5
DL	-95.4	+74.2	-53.0	+31.8	-10.6
LL	-51.8	+43.0	-34.7	+28.9	-23.6
IL	-13.6	+11.3	- 9.1	+ 7.6	- 6.2
Total	-160.8	+128.5	-96.8	+68.3	-40.4

Tabulation of Stresses in Chord Members

Loading	L_6L_2	L_2L_4	L_4L_6	U_1U_3	U_3U_5
	+	+	+	-	-
DL	67.5	157.5	187.5	120.0	180.0
LL	36.6	73.5	84.6	60.8	81.6
IL	9.6	19.4	22.2	15.9	21.4
Total in Kips #	113.7	249.4	294.3	196.7	283.0

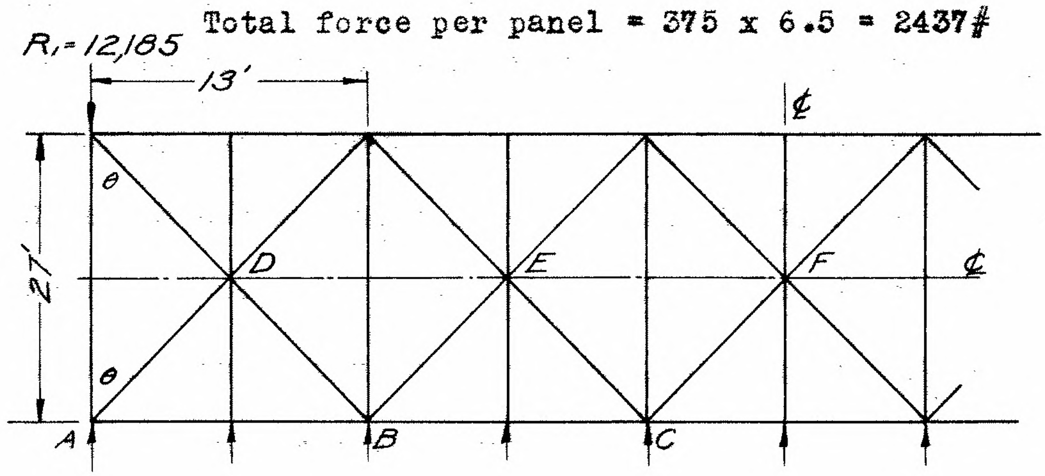
Total Stress in # = Stress in Kip x 1000

Wind Stresses

Assuming that $1\frac{1}{2}$ times the projected area = 4.5 Ft^2 per ft. of span, Force per foot of span = 4.5 x 30 = 135#/Ft. of span.

Force on live load = 8 x 30 = 240#/Ft. of span.

Total force per panel = 375 x 6.5 = 2437#



Tan $\Theta = \frac{13}{27} = .481$ $\quad\Theta = 25^{\circ}40'$ $\quad$ Sec. $\Theta = 1.109$

Stress in AD = 11867 x 1.109 = 13130#

Stress in BE = 9430 x 1.109 = 10430#

Stress in CF = 6993 x 1.109 = 7740#

Designing of the Sections

Working Stresses for Tension, Compression Shear, & Bearing

Tension = S_t = 18000#/in^2

Axial compression, gross section for values of $\frac{L}{r}$ not greater than 140, $S_c = 15000 - \frac{1}{4}(\frac{L}{r})^2$

Where L = length of compression member in inch, r = least radius of gyration of section in inch.

Compression splice material gross section S_c = 18000#/in^2

Shearing stress shop driven rivets S_s = 13500#/in^2

Shearing stress power driven field rivets S_s = 11000#/in^2

Bearing stress for shop rivets S_b = 27000#/in^2

Bearing stress for power driven field rivets S_b = 22500#/in^2.

$\frac{3}{4}$ inch diameter rivets will be used.

All floor beams are 27 in. x 10 in. flange x 98#/Ft. Car I beam.

See page 91

Bottom Chord

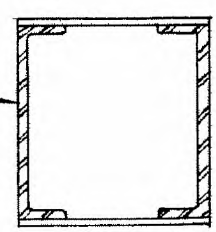
L_0L_2 Required Area = $\frac{\text{Total Stress}}{\text{Working Stress}} = \frac{113700}{18,000}$ = 6.32 in^2

Use 2 - 12 in 25#/Ft.[Gross area = 7.3 x 2 = 14.6 in^2

Net area = 14.6 - (Diam. of hole)(Thickness of web)(No. chan.)

= 14.6 - (.875 x .387)2 = 13.9 in^2.

However, 2 - 12 in. 25#/Ft. channels will be used as shown in the fig.

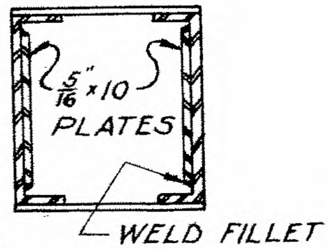


L_2L_4 Required Area = $\frac{249400}{18000}$ = 13.8

Use 2 - 12 in. 25# Chan. Gr A = 14.6

Net A = 17.6 - (.875 x .387)2 = 13.9

L_4L_6 Required Area $= \frac{294,300}{18000} = 16.3$



Using a section of 2 - 12 in. 25# Chan. and 2 - $\frac{5}{16}$ in x 10 plates welded on to the Chan.

Net $A = (14.6 + (\frac{5}{16} \times 10)2 - (.387 + \frac{5}{16}).875 \times 2$

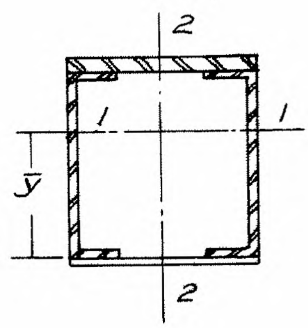
Net $A = 19.6$ in^2.

Design of Top Chord

U_3U_5 Try 2 - 12" - 25# chan. $\frac{3}{8}$ x 12 P.L I_{1-1} chan $= 143.5$

A of Chan. $= 7.3$ A of Pl. $= 4.5$ I_{2-2} chan $= 4.5$

$$y = \frac{7.3 \times 6 \times 2 + 4.5 \times 12.187}{19.14} = 7.45"$$



$$I_{1-1} = (143.5 + 7.32 \times (1.45)^2)2 + \frac{12 \times (.375)^3}{12} + 4.5 \times (4.64)^2 = 414.0$$

$$K_1 = \sqrt{\frac{414}{19.1}} = 4.64$$

$$I_{2-2} = (4.5 + 7.32 \times (5.32)^2)2 + \frac{.375 \times (12)^3}{12}$$

$I_{2-2} = 475.1 > I_{1-1} \therefore K_2 > K_1$

$$K_2 = \sqrt{\frac{475.1}{19.1}} = 4.99$$

$\frac{L}{K} \quad \frac{13 \times 12}{4.64} = 33.6$ $S = 15000 - (\frac{1}{4}\frac{L}{K})^2 = 14733$

Req. $A = \frac{283000}{14733} = 19.2$ Area of section $= 19.14$

Design of Top Chord Cont'd.

U_1U_3 Try 2 - 12" - 25# Chan. and $\frac{5}{16}$ x 12" PL

A of chan. = 7.32 in^2 A of PL = 3.75 in^2

$$\bar{y} = \frac{7.32 \times 6 \times 2 + 4.06 \times 12.156}{18.4} = 7.25 \text{ in.}$$

$$I_{1-1} = (143.5 + 7.32 \times (1.33)^2)2 + 3.75 \times (4.9)^2 \quad I_{1-1} = 409.8 \text{ in}^4$$

$$K_1 = \sqrt{\frac{409.8}{18.4}} = 4.62 \text{ in.}$$

$$I_{2-2} = 4.5 + 7.32(5.32)^2 \times 2 + \frac{.312 \times (12)^3}{12} = 466.2 \text{ in}^4 > I_{1-1}$$

$$K_2 > K_1$$

$$\frac{L}{K_1} = \frac{13 \times 12}{4.62} = 33.8$$

$$S = 15000 - \frac{1}{4}(33.8)^2 = 14{,}715\#/\text{in}^2$$

Required $A = \frac{196{,}700}{14715} = 13.3 \text{in}^2$ Area of sec. = 18.4in^2

This section will be used to simplify fabrication.

L_0U_1 Total stress in this member is 160800# which is less than stress in U_1U_3. The effective length is also less than U_1U_3, but same section will be used as in U_1U_3 to simplify fabrication.

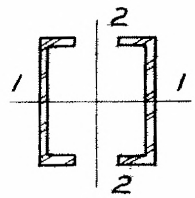
Design of Web Members

U_1L_2 Area required $= \frac{+128500}{18000} = 7.14 \text{ in}^2$

Use 12 in. 32# I Beam.

Gross $A = 9.41 \text{ in}^2$. Net $A = 9.41 - .48 \times .875 \times 4$
$= 7.73 \text{in}^2$

L_2U_3 Length $= 9.2$ ft. Try 2 - 8 in. $11\frac{1}{2}$ #/ft. chan.



$A = 3.36 \text{ in}^2$. $I_{1-1} = 32.3 \text{ in}^4$. $I_{2-2} = 1.3 \text{ in}^4$.

$$I_{1-1} = 32.3 \times 2 = 64.6 \text{ in}^4$$

$$I_{2-2} = (1.3 + 3.36 \times (5.42)^2)2 = 200.2 \text{ in}^4$$

$$K_1 = \sqrt{\frac{64.6}{6.72}} = 3.1 \text{ in.} \quad \frac{L}{K} = \frac{9.2 \times 12}{3.1} = 35.6$$

$$S = 15000 - \frac{1}{4}(35.6)^2 = 14,683\#/\text{in}^2.$$

Required $A = \frac{96800}{14683} = 6.59 \text{ in}^2$.

A of section $= 6.72 \text{ in}^2$.

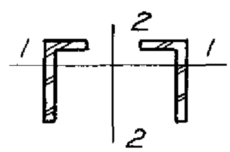
Design of Web Members Cont'd.

U_3L_4 Area required $= + \dfrac{68300}{18000} = 3.8 \text{ in}^2$

Use $- 2 - 5 \times 3 \times \frac{5}{16}$ ∟s $A = 2.4$

Gross $A = 4.8$ Net $A = 4.8 - .31 \times .875 \times 2 = 4.2 \text{ In}^2$

L_4U_5 Try $2 - 5 \times 3 \times \frac{5}{16}$ ∟s $A = 2.4$



$I_{1-1} = 6.3 \times 2$ $K = \sqrt{\dfrac{12.6}{4.8}} = 1.62$

$$\frac{L}{K} = \frac{9.2 \times 12}{1.62} = 67.8$$

$I_{2-2} = (1.8 + 2.4 \times (5.32)^2)2 = 139.4 > I_{1-1}$

$s = 15000 - \frac{1}{4}(67.8)^2$ $s = 13850$

Required $A = \dfrac{40400}{13850} = 2.92 \text{ in}^2$

Net $A = 4.8 - .31 \times .875 \times 2 = 4.2 \text{ in}^2$.

Design of Hanger

LL/panel = 29.3 K

DL/panel = 15.0 K

IL/panel = 29.3 x .263 = 7.7

Total = 52.0 K

Required A = $\frac{52000}{18000}$ = 2.89 in^2. Try 12" 28# I beam.

Net A = 8.23 - .42 x .875 x 4 = 6.7 ⊞

Stress in vertical due to thrust applied at the top chord

R = 150(A + L) where R = Thrust #

A = Area of top chord sect. in^2

L = Panel length in feet.

R = 150(19.14 + 6.5) = 3960#

Moment about the top of the floor beam.

3960 x 51 = 201960 in.#

Maximum stress due to bending = $\frac{201960 \times 6}{183.4}$ = 6600 #/in^2

Direct stress = $\frac{52000}{7.39}$ = 7050 #/in^2

Total stress = 6600 + 7050 = 13650 #/in^2

Use 12" 25# I beam for all hangers and posts.

Laterals maximum stress = 13130.

Required A = $\frac{13130}{18000}$ = .73

Try 3 x 3 x $\frac{5}{16}$ $\underline{/s}$ Gross A = 1.78 in^2

Net A = 1.78 - .31 x .875 = 1.5 Use throughout.

Details

Working Stresses in Rivets

Shearing stress shop driven rivets $S_s = 13500\#/in^2$

Shearing stress power driven field rivets $S_s = 11000\#/in^2$

Bearing stress shop driven rivets $S_b = 27000\#/in^2$

Bearing stress power driven field rivets $S_b = 22500\#/in^2$

Details of Floor Beam Connection

Maximum and shear on the floor beam = 51,500. See page 91.

Rivets on to the truss are field rivets.

$$A \text{ of } \frac{3}{4} \text{ rivet} = .442 \text{ in}^2$$

Number of rivets required $= \dfrac{51,500}{11,000 \times .442} = 10.6$ and

$2 - 3 \times 3 \times \frac{5}{16} L^s$

Number of rivets for bearing $= \dfrac{51500}{22500 \times .75 \times .312} = 8.1$

Use 12 rivets.

Web rivets are shop rivets and in double shear.

Number required $= \dfrac{51500}{13500 \times .442 \times 2} = 4.31$

Web thickness = .5

Number required for bearing $= \dfrac{51500}{27000 \times .75 \times .5} = 5.08$

Use 6 rivets.

Details Cont'd.

Rivets in Lateral Bracing

Number of rivets required in shear $= \frac{13130}{11000 \times .42} = 2.7$

Number of rivets required in bearing $= \frac{13130}{27000 \times .75 \times .31} =$ 2.1 Use 3 rivets.

Joint U_1 For U_1L_1 to develop the section

Number of rivets required $= \frac{6.55 \times 18000}{13500 \times .442} = 19.75$

Use 20 rivets.

For U_1L_2, $\frac{7.47 \times 18000}{13500 \times .442} = 22.5$ Use 24 rivets.

For U_1U_2, $\frac{18.4 \times 14715}{13500 \times .442} = 45.6$ Use 46 rivets.

Number required for bearing $= \frac{18.4 \times 14715}{27000 \times .85 \times .387} = 34.6$

Use 46 rivets.

For L_0U_1, $\frac{L}{4} = \frac{9.2 \times 12}{4.62} = 23.9$ $s = 15000 - \frac{1}{4}(23.9)^2 =$

14857

Strength of the post = 14857 x 18.4 = 273,500#.

Number of rivets required for bearing =

$$\frac{273500}{27000 \times .75 \times .387} = 35$$

Number of rivets required for shear =

$$\frac{273500}{13500 \times .442} = 46 \text{ rivets}$$

The force acting across the joint = 160,800#

Area of gusset plates required $= \frac{160800}{18000} = 8.94 \text{ in}^2$

Assuming the effective width of gusset plate along the joint = 13" and using $\frac{3}{8}$ thick plate area $= 13 \times 2 \times .375 = 9.74 \text{ in}^2$

Details Cont'd.

Joint L_0 Number of rivets required for developed L_0U_1 =

$$\frac{273500}{27000 \times .75 \times .387} = 35$$

For L_0L_1 Number of rivets required for bearing =

$$\frac{13.9 \times 18000}{27000 \times .75 \times .387} = 31.9$$

For L_0L_1 Number of rivets required for shear =

$$\frac{13.9 \times 18000}{13500 \times 442} = 42$$ Use 42 rivets.

L_2U_3 Number of rivets required in shear =

$$\frac{6.72 \times 14683}{13500 \times .442} = 16.5$$

L_2U_3 Number of rivets required in bearing =

$$\frac{6.72 \times 18000}{27000 \times .73 \times .22} = 27.2$$ Use 28 rivets.

U_3L_4 Number of rivets required for bearing =

$$\frac{4.2 \times 18000}{27000 \times .75 \times .31} = 12$$ Use 12 rivets.

L_4U_5 Number of rivets required for bearing =

$$\frac{4.2 \times 18000}{27000 \times .75 \times .31} = 12$$ Use 12 rivets.

Laterals Number of rivets required for bearing =

$$\frac{13,130}{27000 \times .75 \times .31} = 2.1$$

Laterals Number of rivets required for shear =

$$\frac{13,130}{11000 \times .442} = 2.7$$ Use 3 rivets.

Number of rivets required to transmit the load from gusset plates to chords

L_2 $\Sigma F_x = 96.8 \times .707 + 128.5 \times .707 = 159.4$

$\Sigma F_y = 128.5 \times .707 - 96.8 \times .707 = 22.6$

Details Cont'd.

$R = \sqrt{(159.1)^2 + (32.3)^2} = 161.0$ Kp.

Number of rivets required for shear =

$$\frac{161.000}{13500 \times .442} = 27.$$

Number of rivets required for bearing =

$$\frac{162300}{27000 \times .75 \times .375} = 21.2 \quad \text{Use 28 riv.}$$

U_3 $\quad \Sigma F_y = (96.8 - 68.3).707 = 20.1$

$\Sigma F_x = (96.8 + 68.3).707 = 117$

$R = \sqrt{(117)^2 + (20.1)^2} = 118800\#$

Number of rivets required for shear =

$$\frac{118800}{13500 \times .442} = 19.8 \quad \text{Use 20 riv.}$$

L_4 $\quad \Sigma F_y = (68.3 - 40.4).707 = 19.7$

$\Sigma F_x = (68.3 + 40.4).707 = 76.8$

R = 79.3 Kp.

Number of rivets required $= \dfrac{79300}{13500 \times .442} = 13.3$

U_5 $\quad \Sigma F_y = 0$

$\Sigma F_x = (40.4 + 40.4).707 = 57,000\#$

Number of rivets required $= \dfrac{57000}{13500 \times .442} = 9.6$

Rivet Spacing in Cover Plate

Let S_h = increment of flange stress in #/inch.

$S_h = \dfrac{A\ V}{A_2 h}$ Where A_1 = Area of cover plate

V = Total vertical shear on a section

A_2 = Total area of section

h = Effective depth

Details Cont'd.

$$S_h = \frac{.31 \times 12(.75000 + 36600 + .26 \times 36600)}{18.4 \times 34} =$$

$$\frac{3.72 \times 121110}{625} = 720\#/\text{in.}$$

There are two rivets transfering this stress.

$$\text{Pitch of rivets} = \frac{2 \times 11000 \times .442}{720} = 13.5 \text{ in.}$$

But the spacing shall be as follows: A spacing of 3 in. for a distance of 18 in. from each end. Beyond this point the pitch shall be increased by $\frac{3}{4}$ inch until maximum spacing of 6 in. is reached which shall be used for the remaining distance.

Lacing Bars

Lacing of compression members is designed to resist shearing stress normal to the member whose magnitude =

$$V = \frac{P}{100}\left(\frac{100}{\frac{L}{K} + 10} + \frac{\frac{L}{K}}{100}\right)$$

Where P = Allowable compressive axial load on member #,

L = Length of member in inches.

K = Radius of gyration of section about the axis perpendicular to the plane of the lacing in inches.

$$U_3U_5 \quad \frac{283000}{100} \quad \frac{100}{\frac{13 \times 12}{4.99} + 10} + \frac{\frac{13 \times 12}{4.99}}{100} \qquad V = 7750\#$$

$$\text{Axial force on bar} = \frac{7750}{2 \times \cos 30^\circ} = \frac{7750}{2 \times .866} = 4470\#$$

Details Cont'd.

Assume $2\frac{1}{4} \times .31$ bar 60° with long axis   Length of bar $= 11''$.

$I = \frac{2.25(.31)^3}{12} = .00558$ $K = \sqrt{\frac{.00558}{2.25 \times .31}} = .0895$

$\frac{L}{K} = \frac{11}{.0895} = 12.30$

$S = 15000 - \frac{1}{4}(123)^2 = 11225\#/in^2$

Required $A = \frac{4470}{11220} = .397\ in^2$.

Area of section $2.25 \times .31 = .7\ in^2$.

Length of flange included between lacing bar $= 9.2 \times .5 = 4.6''$

K for channel = .79. Therefore $\frac{L}{K} = \frac{4.6}{.79} = 5.83$

Since this is the minimum size bar permissible to use and the stresses in the other members are less, this bar then will be used in all members.

Design of Railing

Uniform load on railing $w = 100\#$ per foot of length.

Length of railing $L = 6.5'$

Maximum bending moment $M = \frac{wL^2}{8} = \frac{100(6.5)^2 \times 12}{8} = 63380\#/in.$

Try $3 \times 3\frac{1}{2} \times \frac{5}{16}$ Ls $S = \frac{6338 \times 2.19}{1.6} = 8670\ \#/in^2$

Vertical End Post for Railing

Load from the lower railing $P_1 = \frac{wL}{2} = \frac{100 \times 6.5}{2} = 325\#$

Load from the upper railing $P_2 = \frac{wL}{2} = \frac{100 \times 6.5}{2} = 325\#$

Details Cont'd.

P_1 acts 2 ft. from support) Cantilever loading
P_2 acts $3\frac{3}{4}$ ft. from support)

$M = (325 \times 2 + 3.75 \times 325)12 = 22500"\#$

Try $4 \times 4 \times \frac{1}{2}$ L

$S = \frac{Mc}{I} = \frac{22500 \times 2.82}{5.6} = 11450\#/in^2$

Design of Bearing Plate for Pier

Loads per truss DL = 75000#

LL = 46100#

IL = 12100#

Total load per truss = 133100#

Total load coming to column of pier = 133100 x 2 = 266200#

Area of plate required $= \frac{266200}{600} = 445$ in^2

Assume column 24 x 24 at the top, therefore

$S_c = \frac{266200}{24 \times 24} = 464\#/in^2$

$M = \frac{wL^2}{8} = \frac{464(24)^2}{8} = 33500"\#$

$\frac{M}{S} = \frac{I}{c} \qquad \frac{bt^2}{6} = \frac{33500}{18000}$

$t^2 = .46$ in. $\qquad t = .68$ in.

Use 1 in. thick 24 x 24 in. plate.

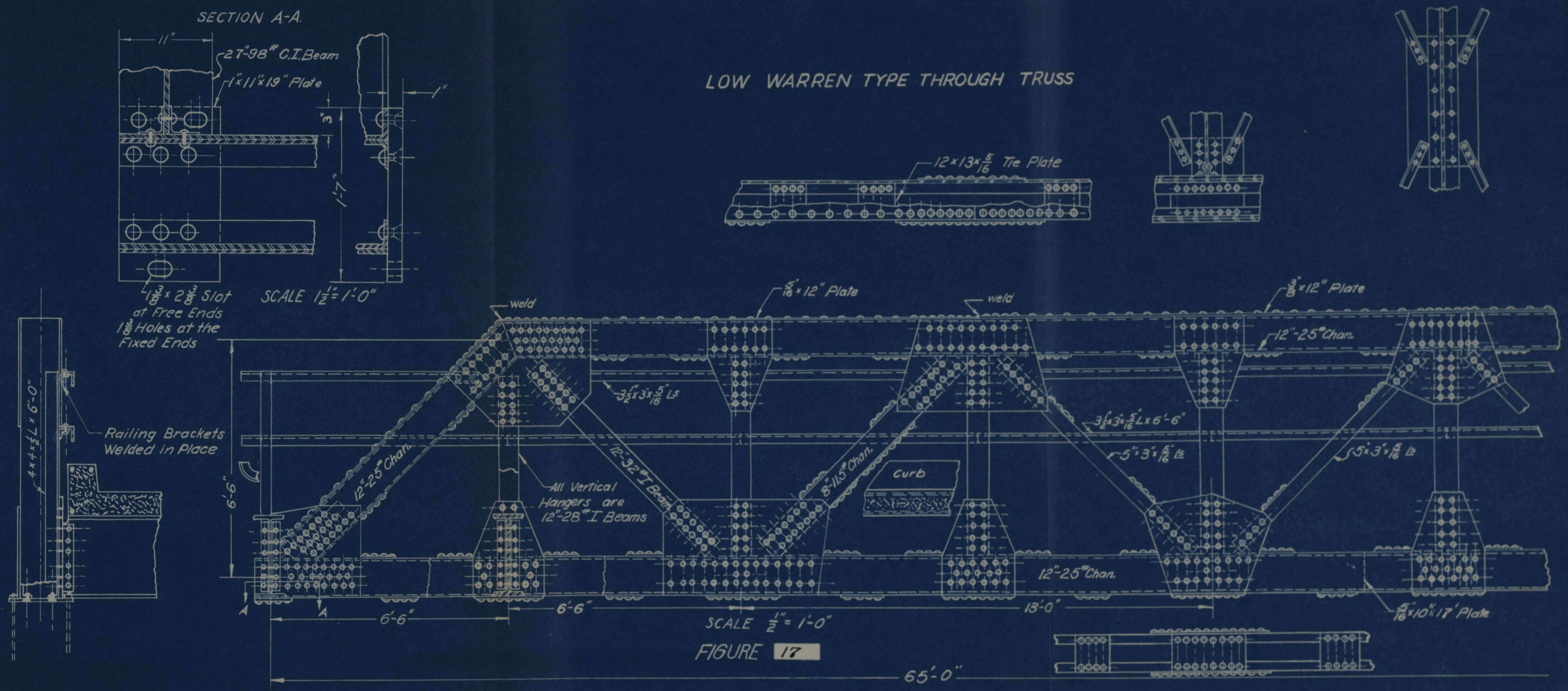


FIGURE 17

Fig. 17

PART IV

MATERIALS AND COST

Tabulation of materials required for construction of

the encased I Beam Bridge

Reinforcing steel in slab

Mark	Size of bars	Length of bars in feet	no. of bars	wt./ft in lbs.	total wt.
MK-S-600	$\frac{3}{4}$ Round	1.75	130	1.502	342
MK-S-601	$\frac{3}{4}$ Round	26	130	1.502	5090
MK-S-500	$\frac{5}{8}$ Round	26	130	1.043	3530
MK-S-450	$\frac{1}{2}$ Square	30	130	.85	3320
MK-SB-200	$\frac{1}{4}$ Round	3.5	1170	.167	685
				Total	15967

Reinforcing steel in railing, posts, and curb

MK-C-400	$\frac{1}{2}$ Round	130	12	.668	1042
MK-R-400	$\frac{1}{2}$ Round	9.5	172	.386	475
MK-R-401	$\frac{1}{2}$ Round	7.4	68	.386	195
MK-C-400	$\frac{1}{2}$ Round	2.5	260	.386	251
MK-C-500	$\frac{5}{8}$ Round	130	12	1.043	1629
				Total	3592

Reinforcing steel in Pier 1 and 2

MK-D-950	$1\frac{1}{8}$ Square	26	1	4.303	112
MK-D-1050	$1\frac{1}{4}$ Square	26	3	5.313	414
MK-F-500	$\frac{5}{8}$ Round	5	20	1.043	104.3
MK-C-1050	$1\frac{1}{4}$ Square	8	25	5.313	1062
MK-C-950	$1\frac{1}{8}$ Square	16	25	4.303	1720
MK-PB-700-1-2	$\frac{7}{8}$ Round	6	26	2.049	319
MK-PB-500	$\frac{5}{8}$ Round	8	21	1.043	175
MK-DV-400	$\frac{1}{2}$ Round	15	21	.386	316
MK-DH-400	$\frac{1}{2}$ Round	26	13	.386	338
				Total	4590.3

Tabulation of materials cont.

Total steel in pier 1 and 2 9180.6 lbs.

Reinforcing steel in pier 3

Mark	size of bars	Length in feet	no. of bars	wt./ft. in lbs.	total wt.
MK-B-700	$\frac{7}{8}$ Round	26	5	2.044	266
MK-D-850	1 Square	26	2	3.4	177
MK-D-1050	$1\frac{1}{4}$ square	26	4	5.313	552
MK-DH-400	$\frac{1}{2}$ round	26	6	.668	104
MK-DV-400	$\frac{1}{2}$ round	7.5	21	.668	105
MK-C-850	1 square	15.5	20	3.4	1050
MK-C-200	$\frac{1}{4}$ round	8	10	.167	12.8
MK-F-1050	$1\frac{1}{4}$ square	26	4	5.313	553
MK-F-1051	$1\frac{1}{4}$ square	28.5	5	5.313	757
MK-F-400	$\frac{1}{2}$ round	5	10	.668	33.2
				Total	3610

Reinforcing steel in Abutment

Mark	size of bars	Length in feet	no. of bars	wt./ft. in lbs.	total wt.
MK-A-400	$\frac{1}{2}$ round	28	5	.386	54
MK-A-450	$\frac{1}{2}$ square	5	27	.85	115
MK-AS-400	$\frac{1}{2}$ round	11.5	55	.668	422
MK-AS-401	$\frac{1}{2}$ round	28	10	.668	187
MK-AV-400	$\frac{1}{2}$ round	3	55	.668	111
				total	889

Reinforcing steel in the two abutments 1,778 lbs.

Total reinforcing steel required 35,169 lbs.

Tabulation of materials cont.

Concrete in slab	90.0 cu.yds.
Concrete in curb	8.75 " "
Concrete in posts	1.65 " "
Concrete in beams	56.6 " "
Concrete in railing	2.8 " "
Concrete in Pier 1 and 2	55.0 " "
Concrete in Pier #3	29.4 " "
Concrete in Abutment	16.3 " "
total	206.5 " "

Tabulation of Material Required for

Construction of the Truss Bridge

Reinforcing Steel in the Abutment

Mark	Size	Length of bar in ft.	No. of bars	Wt. per foot in lbs.	Total weight in lbs.
MK-A-300	$\frac{3}{8}$ Round	5	28	.382	53.5
MK-A-1050	$1\frac{1}{4}$ Square	29	7	5.3	1075.0
MK-AS-400	$\frac{1}{2}$ Round	11.5	55	.682	412.0
MK-AS-401	$\frac{1}{2}$ Round	28	10	.682	191.0
MK-AV-400	$\frac{1}{2}$ Round	3	55	.682	113.0
				Total	1844.5 #

Total Reinforcing Steel Required = 21821.5 #

Tabulation of Material Required for Construction of the

Truss Bridge

Reinforcing steel in one truss

Mark	size of bars	length in feet	no. of bars	wt./ft. in lbs.	total wt. in lbs.
MK-S-600	$\frac{3}{4}$ round	65	25	1.502	2450
MK-S-601	$\frac{3}{4}$ round	70	25	1.502	2640
MK-S-700	$\frac{7}{8}$ round	3.4	250	2.044	1735
MK-S-400	$\frac{1}{2}$ round	25	66	.682	1120
				total	7945

Total reinforcing steel in the slab 15,890

Reinforcing steel in curb

MK-C-500	$\frac{5}{8}$ round	65	12	1	837
MK-C-400	$\frac{1}{2}$ round	2	128	.682	175
				total	1012

Reinforcing steel in Pier

MK-D-500	$\frac{5}{8}$ round	29	2	1.05	61
MK-C-950	$1\frac{1}{8}$ square	23	16	4.31	1585
MK-C-1050	$1\frac{1}{4}$ square	23	8	5.3	975
MK-DH-400	$\frac{1}{2}$ round	28	13	.682	248
MK-DV-400	$\frac{1}{2}$ round	13	25	.682	222
MK-F-500	$\frac{5}{8}$ round	5.25	10	1.05	55
MK-C-200	$\frac{1}{4}$ round	7.5	23	.167	29
			total		3075

Tabulation of Materials Cont'd.

Weight of Truss

Member	Length in feet	Wt./ft. in lbs.	Total wt. in lbs.	Total weight
L_0U_1	8.5	62.8	533	
U_1L_2	7.0	32	224	
L_2U_3	7.4	23	170	
U_3L_4	7.4	16.4	121	
L_4U_5	7.4	16.4	121	
U_1U_2	6.5	62.8	408	
U_2U_3	6.5	62.8	408	
U_3U_4	6.5	65.3	424	
U_4U_5	6.5	65.3	424	
L_0L_1	7.2	50.0	360	
L_1L_2	6.5	50.0	325	
L_2L_3	6.5	50.0	325	
L_3L_4	6.5	50.0	325	
L_4L_5	6.5	50.0	325	
U_1L_1	5.0	25.0	125	
U_2L_2	5.4	25.0	132	
U_3L_3	5.4	25.0	132	
U_4L_4	5.4	25	132	
$U_5L_{5/2}$	2.5	25	68	
				5082

Web Plates in Lower Chord

Member	Length in ft.	Thickness in ft.	Width in ft.	Vol. in ft.	Wt. in lbs.
L_4 L_5	6.5	0.0261	.834	.1415	69.5 #

Bearing Plates

Area ft^2.	Thickness in ft.	Number	Vol. ft^3.	Wt. in lbs.	Total wt. in lb.
4	0.0835	1	.334	163.0	41
1.59	0.0835	1		65.0	32.5

Anchor Bolts

Area	Length	Number	Total wt. in lb.
0.102	1.17	2	24

Total 5,249

Tabulation of Materials Cont'd.

Gusset Plates

At Joints	Area ft^2.	Thickness in ft.	Wt. in lbs.	Total Wt. in lbs.
L_o	13.75	0.0312	210	
L_1	12.38	0.0312	189	
L_2	23.4	0.0312	355	
L_3	12.38	0.0312	189	
L_4	16.0	0.0312	244	
$L_{5/2}$	6.19	0.0312	95	
				1282
U_1	19.0	0.0312	290	
U_2	12.38	0.0312	189	
U_3	18.0	0.0312	275	
U_4	12.38	0.0312	189	
$U_{5/2}$	8.0	0.0312	122	
				1065

Gusset Plates at the Center of Floor Beam for Laterals

Area ft^2.	Thickness in ft.	Vol. ft^3.	Number	Total wt. in lbs.
3.02	0.026	.0785	2.5	96

Gusset Plates on Truss for Laterals

Area ft^2.	Thickness in ft.	Vol. ft^3.	Number	Total wt. in lbs.
4.5	0.026	.117	2	115

Tie Plates

Length	Width	Thickness	Number	Total Vol.	Wt. in lbs.	Total wt. in lbs.
1	1	0.026	38	1.07	522	
						3080

Tabulation of Material Cont'd.

Cover Plate

Length ft.	Width ft.	Thick's ft.	Vol. ft^3	Weight	Total wt.
20.5	1	0.026	0.534	261	
13.0	1	0.031	0.403	198	
					459

Lacing Bars

Length	Width	Thick's	No. bars	Total vol.	Weight	Total wt.
0.92	0.19	0.026	20	0.0907	45	
						45

Rail Posts

Height	Weight/ft.	Number	Weight	Total wt.
5.5	12.75	1	70	
				70

Railing

Length	Weight/ft.	Weight	Total wt.
65	6.6	430	
			430

Rivets

Size	Number	Weight	Total wt.
0.8125	1150	333	
			333

Floor Beams

Length	Weight/ft.	Number	Weight	Total wt.
13	98	5.5	7,000	
				7,000

Joining Angles

Length	Weight/ft.	Number	Weight	Total wt.
1.83	6.1	11	129	
				129

Lateral Bracing Angles

Length	Weight/ft.	Number	Weight	Total wt.
13	6.1	10	792	
				792
			Total	9282

Total per Truss = 35,222#

Assumed Weight per Truss = 40,000#

Encased I Beam Bridge

Total reinforcing steel required	35169#
Cost of reinforcing steel in place	$37.50 per 1000#
Cost of reinforcing steel	$1,318.84
Total concrete required	260.5 Cu. Yds.
Cost of concrete and forms in place	$25.00 per Cu. Yd.
Cost of concrete and forms	$6,512.50
Weight of steel in I Beams	50400#
Cost of steel in place	$50.00 per 1000 #
Cost of steel	$2,520.00
Cost of excavation for piers	$125.00
Total cost of the Encased I Beam Bridge	$10,476.34

Truss Bridge

Reinforcing steel required	21821.5#
Cost of reinforcing steel	$818.30
Concrete required	171.2 Cu. Yds.
Cost of concrete and forms	$4280.00
Structural steel required	140888#
Cost of structural steel in place	$55.00 per 1000#
Cost of structural steel	$7748.84
Cost of excavation for pier	$62.50
Total cost of the Truss Bridge	$12909.64
Difference	$2433.30

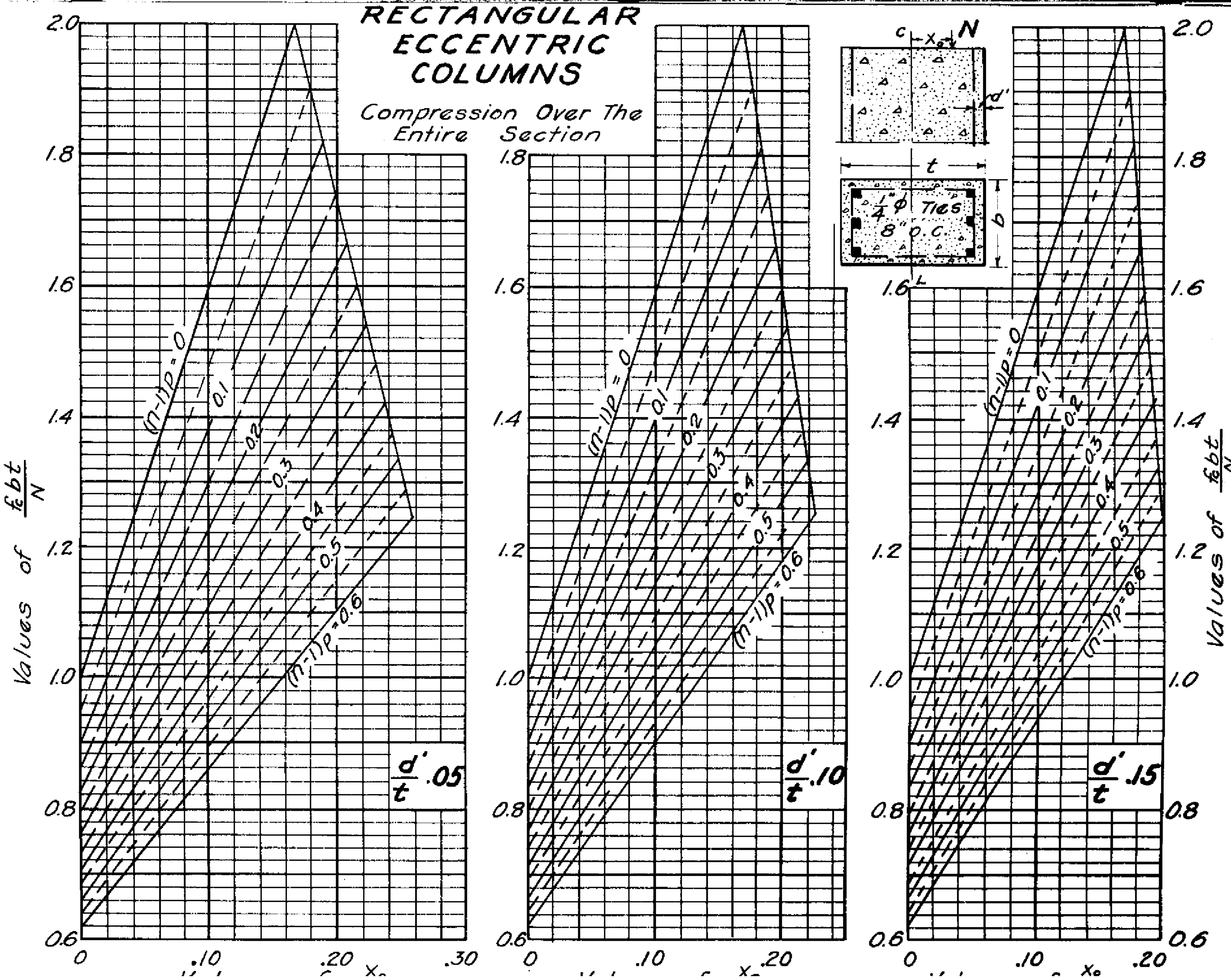
CONCLUSIONS

There are many types of bridges which could have been designed for this particular project. Some of them undoubtedly would cost less to build. The Encased I Beam was selected because of its extensive use in the Southeastern States and the Low Warren Truss because of its similarity in appearance and its popularity in other sections of this country. The visibility from the roadway of both bridges is practically the same and any preference as to the appearance from the side would be largely a matter of personal opinion. The maintenance cost of either bridge would probably be a negligible factor, though the steel truss would require periodic painting. However, the salvage value of the truss may be considered.

Both designs are suitable for present as well as near-future conditions of traffic in this location as estimated by the Georgia State Highway Department. There may be, however, factors unforseen by these experts in traffic expansion which would cause these designs to become prematurely obsolete. It is quite certain that deterioration would not terminate the safe use of either bridge before this possible obsolescence would occur.

The first cost of the Encased I Beam Bridge is $2433.30 less than that of the Truss Bridge. Therefore, if the first cost is the governing factor in the selection of a design, the Encased I Beam Bridge should be used. However, other factors would have a considerable influence on the selection of the type bridge.

The preceding tabulations of costs are based on current prices of material and labor in the Atlanta area for the fall of 1939.


RECTANGULAR ECCENTRIC COLUMNS
Compression Over The Entire Section
Values of $\frac{f_c bt}{N}$
$(n-1)p = 0$
0.1
0.2
0.3
0.4
0.5
$(n-1)p = 0.6$
$\frac{d'}{t}$ .05
$\frac{d'}{t}$ .10
$\frac{d'}{t}$ .15
$\frac{1}{4}$" φ Ties 8" O.C.
c
x_o
N
d'
t
b
0.6
0.8
1.0
1.2
1.4
1.6
1.8
2.0
0
.10
.20
.30

RECTANGULAR ECCENTRIC COLUMNS

Tension over Part of the Section

$f_s = n f_c \left[\frac{1}{k} - 1\right]$